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Impact of dowel bar number and location on pavement performance in a low volume road

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Impact of dowel bar number and location on pavement performance in a low volume road

by

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This is to certify that the master's thesis of

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Signatures have been redacted for privacy

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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ABSTRACT

In jointed concrete pavements, dowel bars are typically used to transfer loads between adjacent slabs. These dowels are typically made of steel and are spaced 12 inches on center for the full length of a transverse joint. The objective of this research was to evaluate the impact of the number of dowel bars and dowel location on pavement performance and joint performance. Consequently, four dowel arrangements were evaluated: 1) zero dowels, 2) three dowels in the outside wheel path, 3) four dowels in the outside wheel path, and 4) full basket of dowels (twelve). In addition, two test sites were prepared with two different subgrades, one with a compacted soil subgrade (Rural site) another with a built up asphalt surface treatment subgrade (Urban site).

Evaluations of the test sections were performed biannually (early fall or late summer and early spring) over a five-year testing period. In addition, a soil investigation was performed using in-situ soil classification from soil borings and consultation of US Department of Agriculture soil survey. Biannual evaluation of both the Urban and Rural sites consisted of: 1) visual distress surveys, 2) joint opening measurements, 3) joint faulting measurements, and 4) deflection measurements using an Iowa DOT (Department of Transportation) Road Rater.

Analysis of the biannual testing indicated that the stiffness of a subgrade magnifies the effect of dowel arrangement on a pavement. It was recommended for pavements with a weak subgrade (dynamic k-values less than 200) to use the standard (full) dowel compliment. For pavements with a strong/stiff subgrade (dynamic k-values greater than 220), three or four dowels in the outside wheel path will suffice. Additional investigation is needed to recommend a dowel bar arrangement for moderately weak to moderately strong subgrades, possibly with dowels in the inside wheel path.

1. INTRODUCTION

1.1 Background

Formal road building began in the United States shortly after the founding of the colonies as early settlers provided for streets in their cities [1]. In the mid to late 1800's several early concrete pavements were built in Europe (Inverness, Scotland as early as 1865, and Grenoble, France in 1876) and in the United States (Bellefontaine, Ohio in 1892, Richmond, Indiana in 1896, and LeMars, Iowa in 1904) [2,3]. Although concrete pavements were in use in the late 1800's, it was not until about 1910 that concrete pavements gained widespread interest [2].

These early concrete pavements were constructed from slabs of concrete, with the small spaces left between adjacent slabs (to be later filled with earth) acting as joints. A concrete pavement constructed in Bellefontaine, Ohio (1892) consisted of small square slabs, five or six feet in length, placed with tarred paper between the slabs to allow for expansion [3].

Joints used to control cracking and to provide for expansion began to appear between 1900 and 1910. These joints were similar to those in the Bellefontaine pavement, wherein the joints were simply small openings between slabs. Other examples of these early joints were found in pavements throughout North America: Grand Rapids, Michigan (1901-1902) with $^1/_4$-inch joints filled with asphalt; Toronto, Canada (1902) with $^3/_4$-inch joints filled with "paving pitch" between 20-foot long square slabs; Richmond, Indiana (1903-1904) one-inch expansion joints; Washington, D.C. (1906) 1-inch joints filled with bituminous material between 100-foot slabs [3].

The use of load transfer devices in joint design appeared soon after the appearance of joints to control cracking and to allow for expansion. Load transfer devices are used to transfer loads from a loaded slab to the adjacent unloaded slab. The use of load transfer devices was first reported in the construction of a concrete pavement near Newport News, Virginia in 1917 [3].

The primary purpose of load transfer devices is to prevent faulting at joints. Load transfer across joints also results in reduced deflection of the loaded slab, reduced stress in the loaded slab, and reduced relative deflection between the loaded slab and adjacent unloaded slab. Although a variety of load transfer devices has been proposed, the two methods most commonly used are aggregate interlock and dowel bars.

1.1.1 Aggregate interlock

Aggregate interlock, which utilizes shear resistance between adjacent slabs, is the simplest means of load transfer [4]. A rough vertical interface between two adjacent slabs of concrete is formed when a crack develops at a joint. Coarse aggregates in the concrete usually remain embedded in either of the crack faces, i.e. the coarse aggregates in one fractured face match the depressions in the other face.

As a tire approaches a joint, the loaded slab deflects vertically, which causes the particles in one face of the crack to come into contact with the other face. Once contact is made, additional shear forces are resisted by a combination of crushing and sliding of the coarse aggregates and the cement mortar. This aggregate interlock provides shear resistance along the crack, thereby transferring load from one slab to another. A recent model proposed by J. C. Walraven [5] suggests that the concrete should be modeled as a two-phase material of aggregate and cement matrix (Figure 1.1).

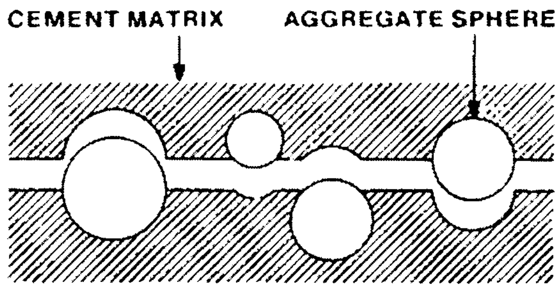

Figure 1.1 Two-phase model of aggregate interlock mechanism. [5]

The long-term effectiveness of aggregate interlock is a function of joint width, load magnitude, aggregate properties, and time of fracture at the joint, with the joint width having the most significant effect. As the joint opening increases, the distance between the aggregate sphere and corresponding cement matrix also increases. This increase in joint opening reduces the ability of the crack faces to resist shear by crushing and sliding of the coarse aggregates and the cement mortar. According to Benkelman [6], a crack with of 0.03 inches resulted in a 50 percent loss in load transfer.

Pavements that make use of aggregate interlock as the load transfer mechanism are more susceptible to pumping, faulting, and reduced load transfer due to weakened soils from moisture fluctuation. Colley and Tayabji [7] report that pavements relying on aggregate interlock experience significant distress in wet climates and are generally used in dry climates in the western United States. Pavements with relatively high truck traffic also experience distress when relying on aggregate interlock. Therefore, another method of load transfer was developed for pavements in wet climates and pavements that experience relatively high truck volumes.

1.1.2 Dowel bars

Dowel bars are typically used in areas with wet climates or wet seasons, where they are used in pavements to transfer load from a loaded slab to the adjacent unloaded slab. The primary purpose of dowel bars is to prevent faulting at joints, but the use of dowel bars across joints also results in reduced deflection of the loaded slab, reduced stress in the loaded slab, and reduced relative deflection between the loaded slab and adjacent unloaded slab. The long-term effectiveness of dowel bars is a function of joint width, load magnitude, base type and depth, construction tolerances, corrosion, dowel looseness, dowel size and spacing, and type of dowel.

The use of dowel bars as load transfer devices was first reported in the construction of a concrete pavement near Newport News, Virginia in 1917. Steel dowels were used in all transverse joints of a concrete pavement constructed near Newport News, Virginia during the winter of 1917-1918 [3]. The steel dowels were used for the purpose of transmitting load across the joint by shear. The steel dowels, $^{3}/_{4}$-inch in diameter, were placed at a spacing of 5 feet (four dowels for a 20 foot wide roadway) in $^{3}/_{8}$-inch joints. Heavy truck traffic during World War I failed to damage the dowel-reinforced joints [3]. Following World War I, the use of steel dowel bars as load transfer devices spread rapidly.

1.1.2.1 Corrosion

The long-term effectiveness of steel dowel bars as load transfer devices is a function of the shear strength and the stiffness of the steel. However, steel is susceptible to corrosion, which can significantly affect the ability of a dowel bar to transfer load and may also induce distress in the pavement. Corrosive agents, e.g. salts and deicers, are brought into contact with to the steel dowel bars through joints and cracks in the pavement. Corrosive agents induce rusting (an expansive reaction) of the steel dowel bars. The volumetric increase of corroded steel leads to increased stresses in the concrete, which in turn leads to cracking and spalling of the concrete above the rusting steel.

Corrosion of the dowel bars can also prevent horizontal movement (e.g. thermal expansion or contraction) of the adjacent slabs, which also leads to spalling and cracking of the pavement at the joint. Finally, the load transfer efficiency of the dowel is reduced due to a decrease in the effective cross-sectional area of the corroded steel dowel bar. To repair the highway pavements in the United States damaged by corrosion, McDaniel [8] estimates that it would cost $212 billion.

1.1.2.2 Dowel looseness

In addition to corrosion, dowel looseness or hollowing also causes distress at a joint. Dowel looseness occurs when voids are present around the dowel bar, which can result from poor construction techniques (e.g. improper consolidation and concrete shrinkage) as well as from damage to the surrounding concrete under repetitive loading [9]. Under repetitive loading, when stress concentrations at the contact between the dowel bar and surrounding concrete exceed the concrete strength, crushing of the concrete occurs. This crushing of the concrete produces a hollow between the dowel bar and surrounding concrete.

Hollowing and dowel looseness reduce the ability of the dowel to resist shear forces; i.e. shear forces in the dowel will develop only after the vertical displacement, caused by the voids, occurs. Using a finite model, the effect of dowel looseness was analyzed by Davids [9]. It was observed that as the gap between the dowel and concrete increased, the shear stress on the dowel bar decreased.

1.1.2.3 Dowel spacing

Standard dowel bar spacing has not yet been developed for dowel bars in concrete pavements, although the practice for most states in the United States is 12-inch spacing [7]. The common practice throughout the United States is to place dowel bars at constant spacing such that a sufficient number of dowels are available to transfer anticipated loads. Colley and Tayabji modeled a pavement with uniform dowel bar spacing of 12 inches across a 12-foot wide pavement section. The model with 12 inch spacing was then compared with the results of a pavement with only six or seven non-uniformly spaced dowel bars (see Figure 1.2 and Figure 1.3).

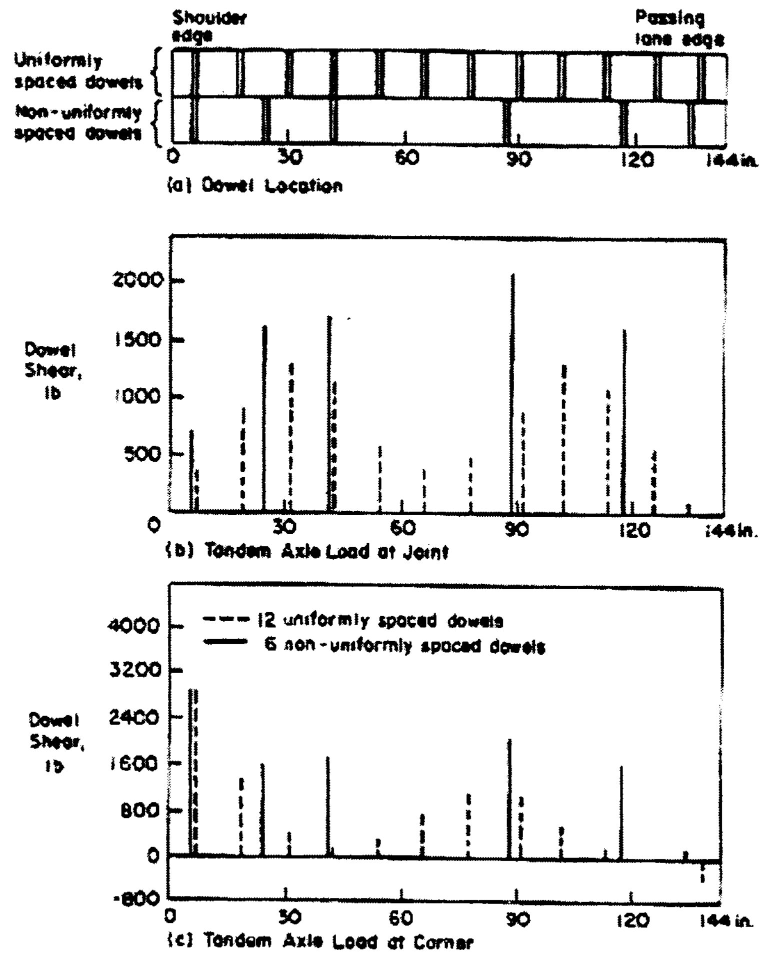

Figure 1.2 Calculated responses for joint with 6 non-uniformly spaced dowels. [7]

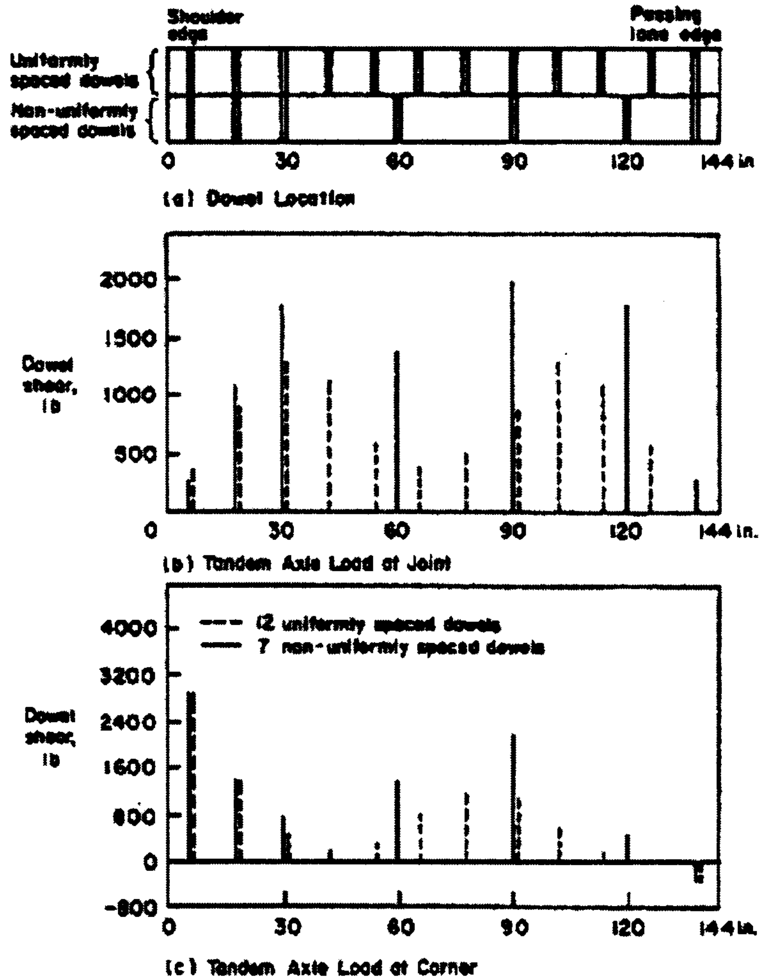

Figure 1.3 Calculated responses for joint with 7 non-uniformly spaced dowels. [7]

The results of these models indicate that applying a tandem axle load at the slab corner for each dowel number and spacing produces almost identical dowel loads. When the load is applied at the joint, a significant difference in dowel load is noted for the different dowel bar spacings. The corner loading condition, however, produced a higher critical dowel loading, consequently controlling the dowel bar design. Colley and Tayabji recommended that non-uniformly spaced dowels be further investigated, but believe that a six dowel system could be appropriate.

1.1.2.4 Current requirements

Current requirements for the use of dowel bars at joints have evolved over time. Load transfer for plain concrete pavements may be achieved by use of aggregate interlock or by dowels [7]. For reinforced concrete pavements, mechanical load transfer devices (e.g. dowel bars) are always specified [7]. Although a variety of mechanical load transfer devises have been proposed, round steel dowel bars are the most widely used devises [7]. According to Colley and Tayabji, current practice for doweled joints in Iowa requires dowel diameters to be $^1/_8$ of slab thickness, the dowel spacing to be 12 inches, and the length of the dowel to be 15 to 18 inches.

1.2 Research Needs

The ability of the pavement to transfer loads to adjacent slabs is pivotal to a quality-performing pavement. Two means of load transfer have been previously discussed: aggregate interlock and dowel bars. Aggregate interlock has been shown to be ineffective in wet climates and/or in roads that experience high volumes of truck traffic, therefore an alternative method of load transfer was needed: dowel bars.

Dowel bar spacing affects the pavement performance and the ability of the dowels to transfer load. As was previously discussed, Colley and Tayabji suggested that pavement performance might not be affected by reducing the number of dowel bars across a pavement section. By reducing the number of dowel bars throughout the joint, the spacing of the existing dowel bars needs to be reconsidered.

Extensive research has been conducted[1] to determine the diameter and spacing of dowel bars as load transfer devices in pavements with heavy truck traffic. The results of this

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1 American Concrete Paving Association, Portland Cement Association, and faculty from the University of Illinois, Michigan State University, and the University of Minnesota.

research have been applied to all levels of traffic with little consideration of the benefits that are being gained as compared to traffic level and construction cost.

Currently, research is being conducted in several field projects throughout the United States. The purpose of this research is to evaluate the constructability and cost-effectiveness of alternative concrete pavement designs, including the number and location of dowel bars in transverse joints, and provide recommendations for alternative concrete pavement design.

Several research projects investigating alternative dowel bar number and spacing are currently being funded by the Federal Highway Administration (FHWA) under the Test and Evaluation 30 program (TE-30), High Performance Concrete Pavement (HPCP), and by the Iowa Department of Transportation (DOT), Testing and Research (TR-420) as well as other state DOTs.

1.3 Research Objective

The objective of this research project is to evaluate the impact of the number of dowel bars and dowel location on pavement performance and joint performance. The impact of four arrangements of dowel bars across a transverse joint is currently being evaluated: 1) no dowel bars, 2) three dowel bars in the outside wheel path, 3) four dowel bars in the outside wheel path, and 4) full basket of dowel bars (i.e. 12 dowel bars). In order to meet the objective of this research project the installation of dowel bars was monitored during construction. Additionally, visual distress surveys and performance evaluations (i.e. midpanel deflection, joint deflection, joint faulting, load transfer, and joint opening) were conducted semiannually after construction.

2. LITERATURE REVIEW

2.1 Abbreviations and Terminology

While conducting research and writing this report, it was necessary to clearly define a variety of key terms. The following section is devoted to listing abbreviations and defining terminology used in this report.

2.1.1 Abbreviations

AADT	Annual average daily traffic
AASHTO	American Association of State Highway and Transportation Officials
ANOVA	Analysis of Variance
DOT	Department of Transportation
FHWA	Federal Highway Administration
ft	Foot
FWD	Falling weight deflectometer
HITEC	Highway Innovative Technology Evaluation Center
HPCP	High Performance Concrete Pavement
HSA	Hollow stem auger
Hz	Hertz
ISU	Iowa State University
lb	Pound
LL	Liquid limit
LT	Left
mm	Millimeters
mil	One thousandth of an inch

NDT	Non-destructive testing
PCC	Portland cement concrete
pci	Pounds per cubic inch
PI	Plasticity index
psi	Pounds per square inch
RT	Right
SHRP	Strategic Highway Research Program
STA	Station
TE	Test and Evaluation project
TR	Testing and Research project
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture

2.1.2 Terminology

- Analysis of variance, a statistical analysis where the difference in the means of two or more variable are determined to be "statistically significant (or not) by comparing the variability within the samples" [10, p. 771]

- Annual Average Daily Traffic, measurement representing the total number of vehicles passing a given location, based upon 24-hour counts taken over an entire year.

- Area, A, cross-sectional "area" of the deflection basin, calculated using the deflection measurements recorded by the four sensors on the Road Rater

- Bearing capacity, load per unit area, where the ultimate bearing capacity is the load per unit area at which a sudden failure occurs in the soil supporting the load

- Confidence interval, the upper and lower limit for the difference between two variables, with 95% probability (for this project)

- Deflection basin, curve formed by deflection responses at known locations away from an applied load

- Dense liquid foundation, force-deflection relationship that is characterized by an elastic spring

- Dynamic loading, loading conditions that represent a situation where the load applied is in constant motion

- Level of significance, p-value, weight of evidence for rejecting the null hypothesis

- Load transfer, ratio of the strain on the unloaded side of the joint to the total strain (sum of the strain on the loaded and unloaded sides) expressed as a percentage

- Measured joint efficiency, ratio of deflections of the unloaded slabs to the loaded slabs

- Modulus of subgrade reaction, k-value, a measure of the stiffness of the subgrade, which equals the ratio of the pressure of a loaded plate (10 psi) to the deflection of the plate, expressed in pounds per cubic inch (pci).

- Non-destructive testing (NDT), testing which results in no major structural damage of the pavement

- Static loading, loading conditions that represent a situation where the load applied is at rest or moving with a constant velocity in a straight line

2.2 Joint Related Distresses

Joint related pavement distresses develop from deficient design, construction, or maintenance practices. Distress requiring repair are primarily caused by [7]:

1. Poor slab support
2. Erodible support
3. Excessive water
4. Particle infiltration into joints
5. Poor load transfer (e.g. from dowel looseness)
6. Excessive traffic loads
7. Sealant failure
8. Long slabs
9. Dowel misalignment
10. Dowel corrosion
11. Soil movements (e.g. pumping)
12. Poor aggregate performance (expansive concretes)

Joint related distresses are usually manifested at or near joints, but may occasionally occur away from joints (e.g. mid-slab cracking). The principal types of joint related distresses (illustrated in Figure 2.1) are [7]:

1. Cracking (transverse, restraint, corner, and D-cracking)
2. Faulting
3. Spalling
4. Raveling
5. Pavement movement.

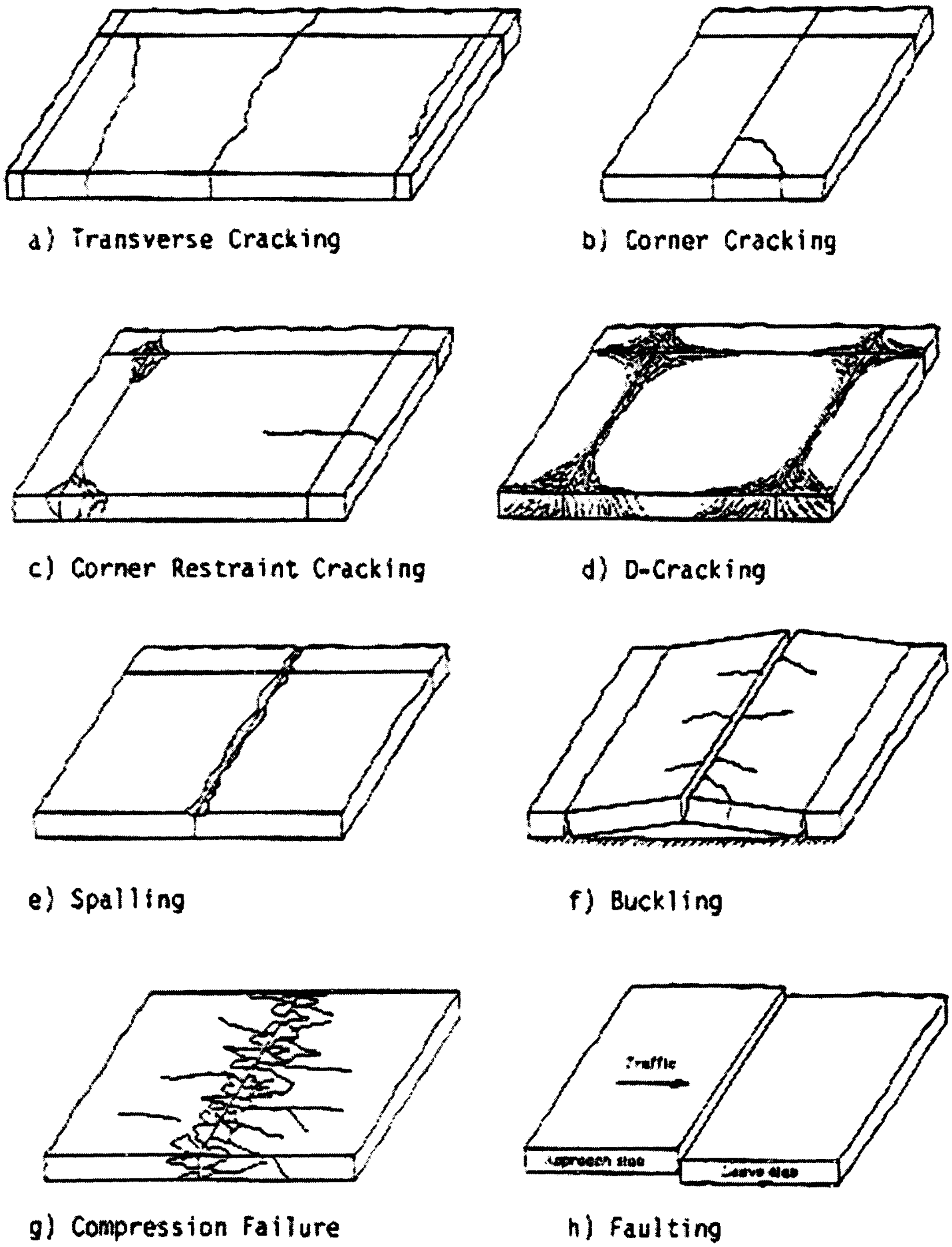


Figure 2.1 Joint related distresses. [7]

The remainder of this section will focus on the causes of cracking, faulting, and spalling; pavement distresses which are related to dowel-reinforced concrete joints.

Cracking of a slab may take a variety forms (e.g. transverse, corner, restraint, D-cracking). Mid-slab cracking in reinforced concrete is anticipated in design [7]. These cracks will remain tight unless a adjacent transverse joint becomes frozen, where the cracks will widen. Corner cracking, in contrast, can be caused by poor subgrade support, lack of adequate load transfer at a joint, excessive loading, or some combination of the three. Restraint cracking is caused by high compressive forces at the joint and results in breaking up of the concrete in the affected area [7]. D-cracking is due to coarse aggregate that is susceptible to freeze-thaw damage [7].

Faulting in pavements is the development of an elevation differential at a joint and is caused by traffic. An erodible subgrade must be present for faulting to occur [7]. Faulting in pavements is minimized by using dowels to transfer loads at joints, providing subsurface drainage, using tied concrete shoulders, and providing good joint sealing. Spalling of the pavement, in contrast, occurs when excessive stress is built up at the joint face, inducing cracking of the concrete.

2.3 Factors Influencing Dowel Behavior

Dowel bars are typically used in areas with wet climates or wet seasons, and in areas with soils susceptible to freeze/thaw cycles or frost action. Dowels are used in pavements for the purpose of transferring load between slabs by shear. The primary purpose of dowel bars is to prevent faulting at joints, but the use of dowel bars across joints also results in reduced deflection of the loaded slab, reduced stress in the loaded slab, and reduced relative deflection between the loaded slab and adjacent unloaded slab. The long-term effectiveness of dowel bars is a function of joint width, load magnitude, base type and depth, construction tolerances, corrosion, dowel looseness, dowel size and spacing, and type of dowel. This section will focus dowel bar placement tolerances, dowel looseness, and resistance to movement (i.e. locked or frozen dowels).

2.3.1 Placement tolerances

The construction process and dowel bar installation can significantly affect the effectiveness of load transfer and can also induce distress in the pavement. As of 1980, the Federal Highway Administration (FHWA) did not specify limits on dowel bar misalignment, but cautioned that dowel placement was extremely important in the proper functioning of the slab and for ling-term performance [11]. Current requirements state that dowels be placed "as parallel as practical to the longitudinal axis and the horizontal plane of the pavement" [7, p. 26]. Presently there are two methods for placing dowel bars in concrete pavement: using supporting assemblies and implanting the dowels. This section will focus on misalignment due to basket assemblies.

Dowels placed using basket assemblies must be fastened securely to the subbase by special nails, stakes, or clips so as not to shift when a paver passes over the assembly. In order to minimize dowel bar movement in the basket assembly, dowels are securely fastened or welded at alternate ends of the basket assembly (to prevent frozen dowels).

Although measures are taken to minimize dowel bar misalignment, dowel bars can be misaligned. When using basket assemblies, dowel misalignment is attributed to several factors, including [7]:

- Basket rigidity
- Quality control during basket fabrication

- Care taken during basket assembly, transportation, and placement
- Fastening baskets to subgrade
- Location of sawcut over basket
- Paving operation
- Field inspection (or lack thereof)

Dowel bars may be misaligned by horizontal, longitudinal, and vertical translation and can also be skewed horizontally and vertically. Misalignment of dowels may cause locked or frozen dowels, which leads to spalling at the concrete face and around the dowel. The different types of dowel misalignment and their possible effects on pavement behavior are illustrated in Figure 2.2 and listed in Table 2.1.

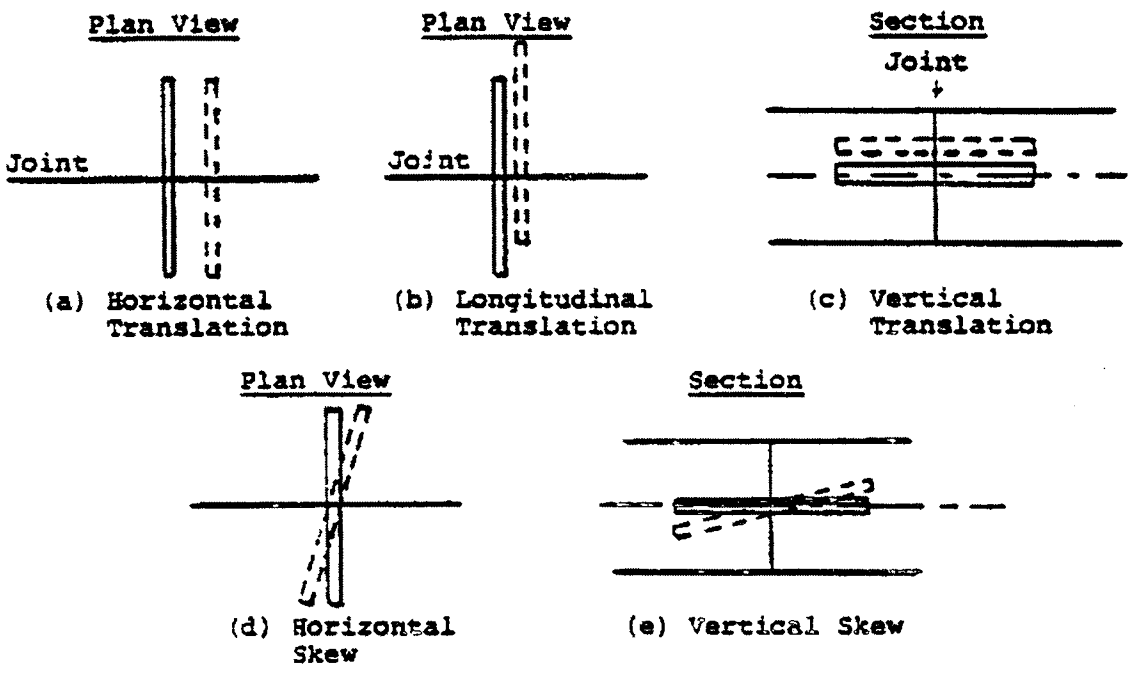


Figure 2.2 Types of dowel misalignment. [7]

Table 2.1 Effects of dowel misalignment on pavement behavior. [7]

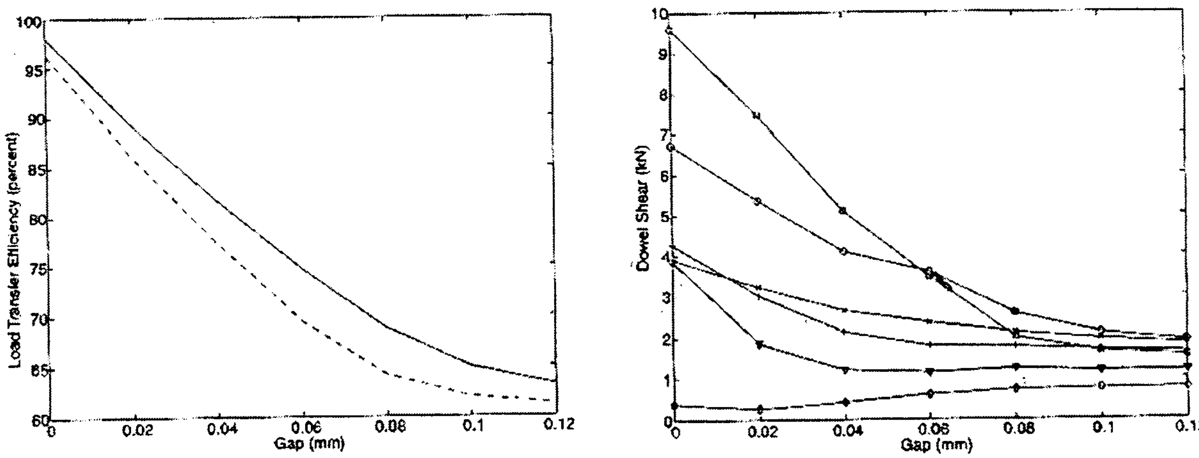
Type of misalignment	Effect on		
	Spalling	Cracking	Load transfer
a) Horizontal translation	--	--	Yes [a]
b) Longitudinal translation	--	--	Yes [a]
c) Vertical translation	Yes	--	Yes [a]
d) Horizontal Skew	Yes	Yes	Yes
e) Vertical Skew	Yes	Yes	Yes

[a] Effect depends on amount of translation

2.3.2 Dowel looseness

In addition to dowel bar misalignment, dowel looseness also influences the load transfer efficiency of dowel bar. As previously discussed, dowel looseness occurs when voids are present around the dowel bar. These voids may result from poor construction techniques or from damage to the concrete surrounding the dowels under repetitive loading. Dowel looseness reduces the ability of dowels to resist shear forces; i.e. shear forces in the dowel will develop only after the vertical displacement, caused by voids, occurs.

The effect of dowel looseness was analyzed using a finite model by Davids [9]. It was observed that as the gap between the dowel and concrete increased, the shear stress on the dowel bar decreased (Figure 2.3.).



a) b)

Figure 2.3 a) Load transfer efficiency and b) dowel shear vs. gap with axial loading. [9]

2.3.3 Slab end movements

The behavior of concrete pavement is a function of the properties of the concrete as well as the subgrade. This section will focus on temperature and moisture induced slab end movements as well as restraint mechanisms.

2.3.3.1 Causes

In addition to deforming under applied stress, like other materials concrete also expands and contracts with changes in temperature and moisture content. Due to the properties of concrete and environmental changes, concrete pavements can expand and contract with time of day, with seasons, and with variations in weather [12]. Additionally, daily and seasonal changes can induce curling and warping of slabs due to temperature and moisture differences between the top and bottom of a slab.

Slab end movements are the result of daily and seasonal temperature variations as well as changes in moisture content of the concrete. A concrete pavement tends to contract as it dries or cools and tends to expand as the pavement warms. Slab end movements associated with temperature variations are a function of the thermal coefficient of the concrete.

Seasonal slab end movements occur over relatively long periods of time, therefore it can be assumed that subgrade friction, due to seasonal movement, is negligible [7]. Daily movements, however, occur over a relatively short period of time, and therefore are affected by subgrade friction.

In addition to daily and seasonal changes in moisture, drying of fresh concrete also causes the concrete to shrink. The amount of drying shrinkage is a function of the amount of mixing water, the water to cement ratio, aggregate type, and curing conditions.

Slab end movements affect the joint design and performance of concrete pavements. In addition to governing joint sealant design and selection, the magnitude of slab end movement also affects load transfer at joints. The load transfer of plain concrete pavements (i.e. pavements relying on aggregate interlock) is affected to a greater extent than in doweled pavements [7].

2.3.3.2 Frozen dowels

The degree to which a joint is frozen, i.e. the slabs are restrained from moving, is a function of dowel bar misalignment and corrosion of the dowel. (Misalignment is discussed in section 2.2.1 and corrosion is discussed in section 1.1.2.1.) Transverse cracking, spalling of the concrete around the dowel, and rupturing of the dowel may result from frozen joints. Frozen joints can be verified by measuring joint width for change over time and by dowel pull out tests [13].

If a dowel or joint should become frozen, the joint may be replaced, or the dowels may be retrofitted (i.e. strip the top half of the pavement, replace the frozen dowels then place new pavement over the dowels). Therefore, to reduce the cost to repair damage to the pavement caused by frozen dowels, preventative measures are needed. To minimize the possibility of frozen dowels more care should be taken in the alignment of dowel bars and, in areas where deicing salts are used, corrosion resistant dowels should be used [13].

2.4 Subgrade

The type of base is an important factor in the performance of jointed concrete pavements. Faulting, cracking, and spalling are all affected by the base layer due to the support provided to the slab and the effect of the base course on the subsurface drainage and erosion potential [14].

2.4.1 Modulus of subgrade reaction

The modulus of subgrade reaction, k-value, is a measure of the stiffness of the subgrade and is a function of soil type, degree of saturation, distance to bedrock, water table, and other factors [15]. The use of a singe k-value in analysis assumes that the subgrade or subbase is elastic [14]. Typically, static k-values range from approximately 50 pounds per cubic inch (pci) for very poor soils to 225 pci for the very good materials; for very good soils, the k-value may reach or exceed 700 pci. [14].

Barenberg and Darter stated that the "placement of a subbase will usually increase the k-value of the total foundation (i.e. subbase and subgrade)" [15, p. 36]. Therefore, a pavement system with an asphaltic pavement base course (Urban site, see section 3.3) would have a higher k-value than a pavement without the asphalt base course (Rural site, see section 3.3). A k-value of at least 100 pci during any month throughout the year must be obtained to provide adequate structural support for a slab, to minimize deflection of a slab, and reduce the potential for joint faulting [15].

2.4.2 Effect of k-value on pavement response

Darter developed a fatigue analysis procedure for plain jointed concrete using a finite element method. The finite element program evaluated the effect of several pavement factors on the critical stress in a PCC (Portland cement concrete) slab. These factors included: 1) slab thickness, H, 2) k-value, 3) thermal gradient, G, 4) slab length, L, and 5) erodibilty of support, ES. This section will focus on the relationship between the k-value and the critical stress (edge stress) as a function of slab thickness, slab length, thermal gradient, and erodibilty of support.

Figure 2.4 illustrates the relationship between the k-value and edge stress as a function of slab thickness, slab length, thermal gradient, and erodibilty of support. The effects of the modulus of subgrade reaction on the total edge stress are summarized below.

- For a given pavement thickness and/or slab length, the edge stress decreases with increasing subgrade stiffness (k-value).
- The slab length has negligible effect on the relationship between the k-value and edge stress.
- A thicker pavement is required for a lower k-value to obtain the same edge stress as a thin pavement with a higher k-value.
- The effect of the k-value on the edge stress for thick slabs is less than for thin slabs.
- As the k-value increase, combined load and curl stresses decrease, with the exception of a daytime thermal gradient where the stress increases with an increasing k-value.

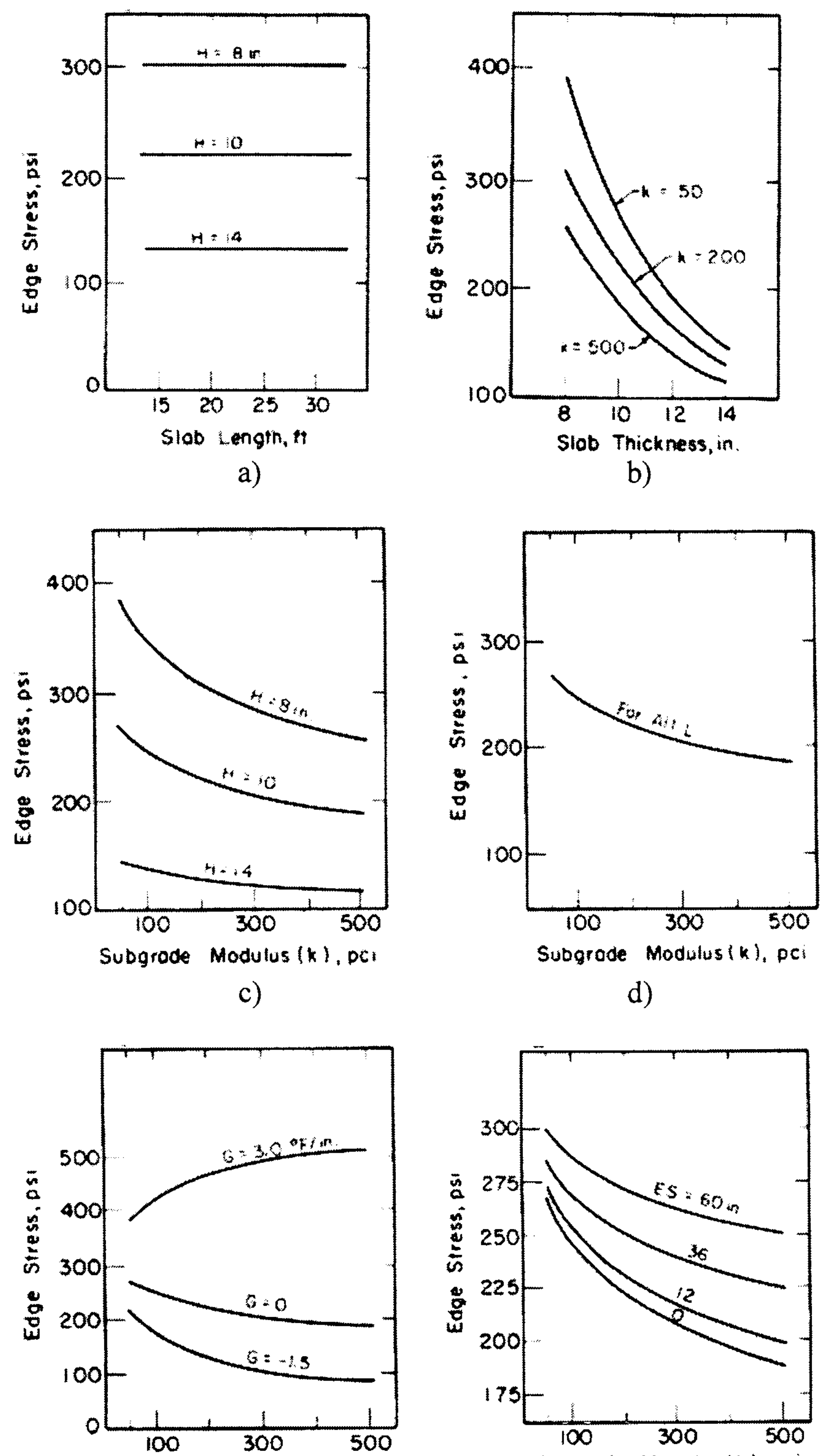


Figure 2.4 Effect of modulus of subgrade reaction on edge stress. [13]

2.4.3 Seasonal effect on the k-values

Pavements located in areas with annual freeze-thaw cycles can be subjected to significant moisture and temperature gradients in the subgrade. The pavement is not only subjected to traffic-induced stress, but also stress produced as a result of the moisture and thermal gradients in the subgrade. During the winter water is drawn upwards from shallow water tables by capillary action and is converted to ice lenses. These ice lenses expand as water is drawn up and frozen to create differential stresses on the overlying pavement. During the spring the ice lenses melt, which causes the subbase to become saturated. In addition, moisture can be introduced into the subsurface layers from infiltration and melting snow from the surface. As a result of the increase in moisture, the bearing capacity of the subgrade may be reduced, which would increase the pavement stresses [16].

Seasonal changes in subgrade moisture content affect the modulus of subgrade reaction. In the spring the subgrade is typically wet with a low k-value, creating a weak foundation to the pavement. The subgrade in the early fall, in contrast, is typically dry with a higher k-value and stronger pavement foundation.

2.5 Road Rater

2.5.1 Introduction

Prior to the use of the Road Rater for non-destructive testing, a pavement and its subgrade were evaluated by coring through the pavement, which was destructive to the pavement. The core was then visually inspected or inspected with a microscope to view details not visible to the naked eye. The subsurface layers could then be analyzed by conducting in-situ testing (e.g. dynamic cone penetrometer) or by obtaining samples of the subgrade and performing laboratory tests.

Non-destructive methods were developed to minimize the damage to a pavement system during testing and to reduce testing time. Testing pavement with a Road Rater allowed a larger number of test sites to be evaluated, as compared to the destructive coring methods, due to the quicker testing procedure. In addition, the Road Rater minimized the damage to the pavement system.

The Road Rater is typically a trailer-mounted testing machine used to measure the deflection of a pavement in response to a known applied load. The Road Rater is pulled along the pavement behind a tow vehicle until a test site is reached, upon which the Road Rater is stopped. A large mass is hydraulically lowered to the pavement and oscillated through a servo valve to produce a loading force. The loading force varies from 400 to 2,400 pounds for rigid pavements [17].

For photos and a schematic of the Road Rater used for this research project refer to Figure 3.8, Figure 3.9, Figure 3.10, and Figure 3.11 (section 3.4.4).

The concrete pavement as well as the subsurface that supports it (i.e. subgrade, subbase, and base) deform when loads are applied and rebound when the load is removed. Therefore, the load applied by the Road Rater deforms both the pavement and subgrade. The combined deflection is measured by four velocity sensors on the Road Rater and is collected by a computer in the tow vehicle. One velocity sensor is positioned directly beneath the load, while the other three sensors are positioned at one-foot intervals. In addition to the four pavement sensors, another velocity sensor is mounted on top of the hydraulic ram and measures the amplitude (peak to peak) mass displacement.

The deflection information obtained by the Road Rater is analyzed to provide an indication of the performance of the pavement. The analysis of joint deflections and midpanel deflections are based on Westergaard's plate theory of a linear elastic, homogeneous, and isotropic material resting a dense liquid foundation.

2.5.2 Deflection basins

The deflection of the pavement at the joint provides information on the load transfer efficiency, which is defined as the pavement's ability to transfer loads from a loaded slab to an adjacent unloaded slab. Load transfer is calculated from the deflections recorded from velocity sensors spaced evenly across a joint. Specifically, the sensor under the load is positioned approximately 6 inches on one side of the joint and second sensor is located approximately 6 inches on the opposite side of the joint. Therefore, if the deflections on either side of the joint are equal, there is 100% load transfer. Lower load transfer results in a larger force on the loaded slab, which amplifies fatigue in the pavement.

Midpanel deflections (i.e. near the center of the slab), on the other hand, provide information on the pavement and subgrade stiffness. In addition to providing information on stiffness of the pavement system, midpanel deflections allow a deflection basin to be defined. The deflection basin is a function of the maximum deflection and area, which is defined as the cross-sectional "area" of the deflection basin.

The shape of the deflection basin can indicate the strength of layers in the pavement system. A point load applied to a weaker pavement, for example, will result in a cone shaped deflection basin with high deflections centralized near the load. A stronger pavement, in contrast will exhibit a broader cone-shaped deflection basin, with the pavement deflecting less near the load and more further away. Possible shapes for a deflection basin are illustrated in Figure 2.5.

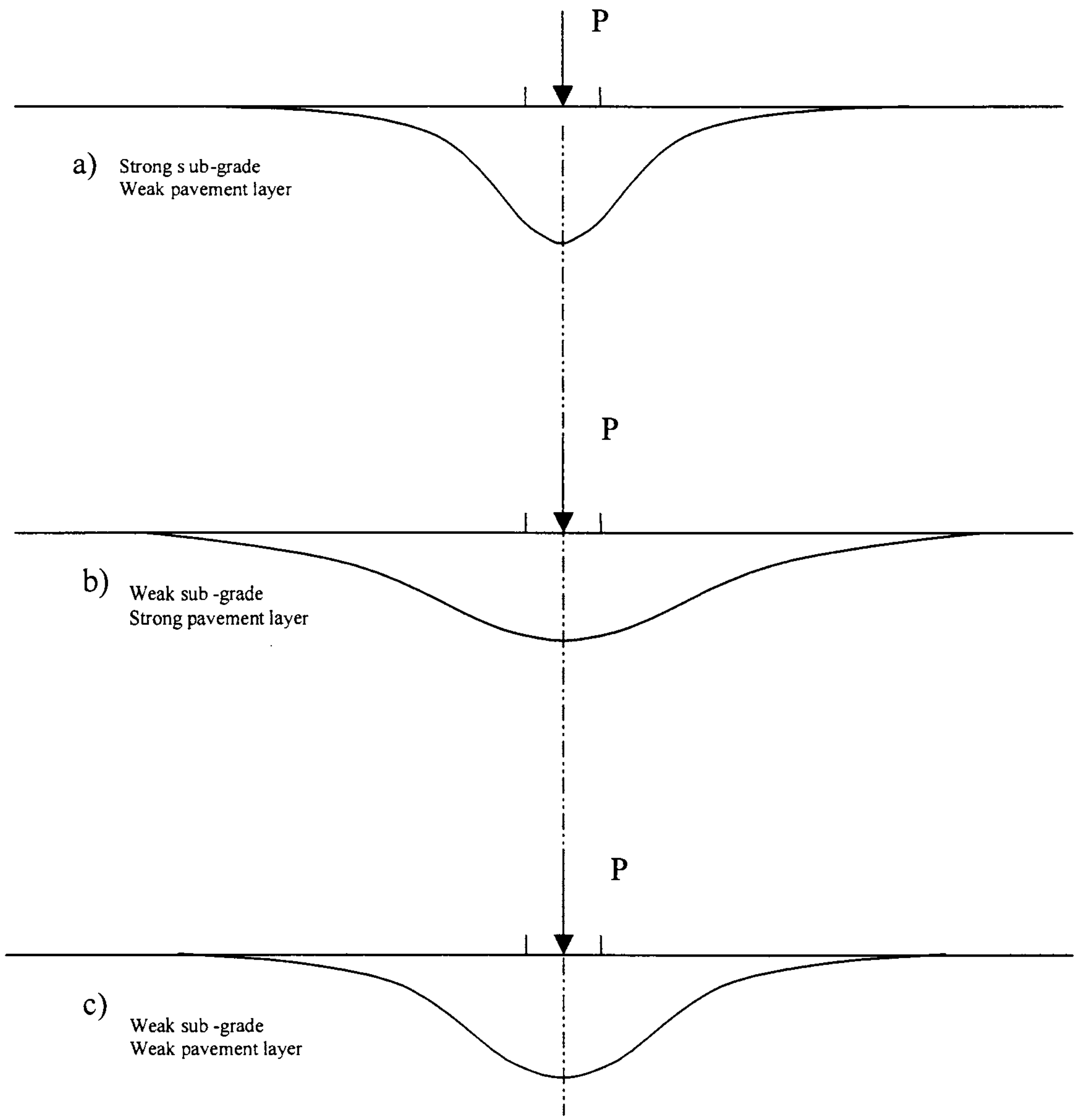


Figure 2.5 Possible shapes of a deflection basin. [8]

The shape of a deflection basin is a function of several factors including slab thickness, slab stiffness, stiffness of the subsurface layers, and weight of the applied load. In addition to properties of the pavement system, changes in the environment and conditions of the subsurface layers also affect the shape of the deflection basin. As a result, the shape of the deflection basin varies seasonally, from pavement to pavement and along a single pavement [18].

2.6 Other Research Projects

Several research projects investigating alternative dowel bar number and spacing are currently being funded by the Federal Highway Administration (FHWA) under the Test and Evaluation 30 program (TE-30), *High Performance Concrete Pavement* (HPCP), and by the Iowa Department of Transportation (DOT), Testing and Research (TR-420) as well as by other state DOTs.

This section will focus on two projects investigating nonuniformly spaced dowels. The first project, conducted by the Iowa DOT, was a laboratory study. The second project, conducted by the FHWA, is a field study.

2.6.1 Theoretical studies

Common practice throughout the United States is to place dowel bars at constant spacing such that a sufficient number of dowels are available to transfer anticipated loads. The most serious faulting in a pavement is located in the outside corner of the outside traffic lane [13]. Implying that the outside edge of the pavement is the location most susceptible to damage. Colley and Tayabji modeled a pavement with uniform dowel bar spacing of 12 inches across a 12-foot wide pavement section. The model with 12 inch spacing was then compared with the results to a pavement with only six or seven non-uniformly spaced dowel bars (see Figure 1.2 and Figure 1.3).

The results of these models indicate that applying a tandem axle load at a slab corner for each dowel number and spacing produces almost identical dowel loads. When the load is applied at the joint, a significant difference in dowel load is noted for the different dowel bar spacings. The corner loading condition, however, produced a higher critical dowel loading, consequently controlling the dowel bar design. Colley and Tayabji recommended that non-uniformly spaced dowels be further investigated, but believe that a six dowel system could be appropriate.

2.6.2 Field studies

The Highway Innovative Technology Evaluation Center (HITEC) initiated TE-30 in May 1992. TE-30, funded by the FHWA, was developed to investigate innovative PCC design and construction. Several areas of investigation for the program include [19, p. 1]:

- Increasing the service life.
- Decreasing construction time.
- Lowering life-cycle costs.
- Lowering maintenance costs.
- Constructing ultra-smooth ride quality pavements.
- Incorporating recycled or waste products while maintaining quality.
- Utilizing innovative construction equipment and procedures.
- Utilizing innovative quality initiatives.

Under TE-30, twenty-two projects have currently been funded by the FHWA, with sites in Illinois, Iowa, Kansas, Maryland, Michigan, Minnesota, Mississippi, Missouri, New Hampshire, Ohio, South Dakota, Virginia, and Wisconsin. These projects are presented in a report submitted to the FHWA [19].

In the summer of 1997, Wisconsin DOT constructed two experimental concrete pavements on Highway 29. These projects were constructed to investigate the constructability and effectiveness of alternative concrete pavement designs [20]. Variables considered in this investigation are [20]: 1) reduced number dowel bar across transverse joints, 2) alternative dowel bar material, and 3) variable pavement cross-sectional thickness. Both projects are summarized below.

2.6.2.1 Wisconsin 2

The Wisconsin 2 project was constructed in September 1997 and consisted of an 11-inch pavement with variable transverse joints of 17-20-18-19 feet. The reinforcement of the pavement consisted of alternative dowel bar material and alternative dowel layouts. The 1.5-inch dowels were placed in the plastic concrete by an automatic dowel bar inserter. The alternative dowel bar layouts, illustrated in Figure 2.6, were selected to reduce the number of dowels required while maintaining the standard placement locations used in Wisconsin [20].

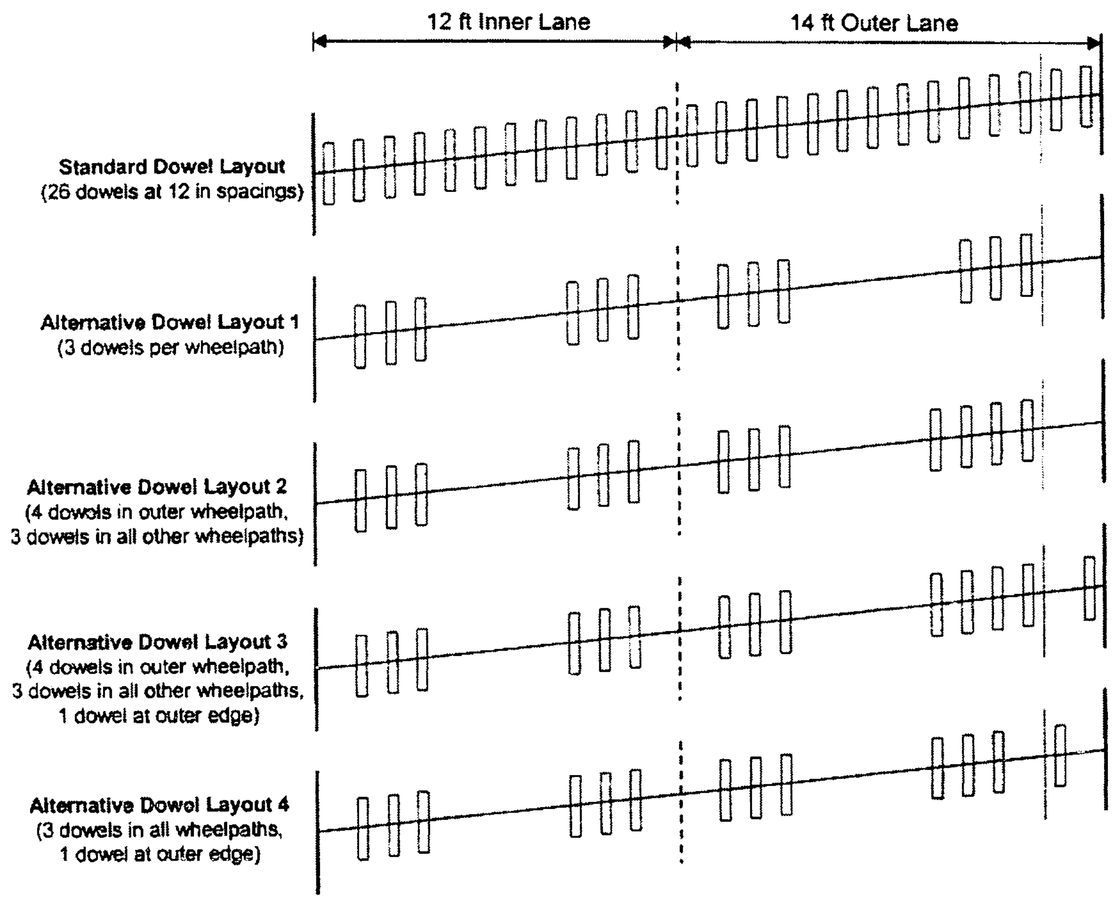


Figure 2.6 Alternative dowel bar layouts for Wisconsin 2 and Wisconsin 3. [19]

Wisconsin DOT and Marquette University monitored the performance of both Wisconsin 2 and Wisconsin 3. The monitoring included [20]: 1) dowel bar study was conducted two months after construction, 2) FWD (falling weight deflectometer) testing, 3) distress surveys, and 4) ride quality surveys.

The dowel bar study detected that the dowel bars were slightly deeper than the mid-depth of the slab and some vertical skewing of the dowels had occurred across the joints. The results from the FWD testing were fairly consistent over time. Some spalling, chipping and fraying of the transverse joints (partially attributed to saw cutting) was observed during the distress surveys. Most of the test sections are performing comparably to the control sections in the ride quality studies.

2.6.2.2 Wisconsin 3

The Wisconsin 3 project was constructed with a variable cross-sectional thickness. The slab was eight inches at the edge of pavement in the passing lane slab that transitioned into 11 inches at edge of pavement in the driving lane. The pavement reinforcement consisted of alternative dowel bar material and alternative dowel layouts. The 1.5-inch dowels were placed in baskets and secured to the subgrade prior to paving. One section contained an alternative dowel bar arrangement (Alternative Dowel Layout 1, Figure 2.6).

3. RESEARCH PLAN

3.1 Introduction

As stated in section 1.3, the objective of this research project is to evaluate the impact of number of dowel bars and dowel location on pavement performance and joint performance. In order to satisfy this objective, a full-scale field application under normal operating conditions was used to compare various pavement and joint characteristics. Evaluation of the performance of alternative dowel bar number and location is part of a five-year study being conducted through a combined effort by Iowa State University (ISU), the Iowa DOT, Union County, and the City of Creston. A thorough comparison of the alternative number and location of dowel bars in concrete pavements is best accomplished over the service life of the pavement. The service life of a pavement can extend over 20 years; therefore, in order to determine the advantages and disadvantages of the alternative number of dowels and dowel location, continuous evaluation is needed.

3.2 Site Description

The Fred Carlson Co. of Decorah, Iowa constructed two separate test sites in Union County near Creston, Iowa in August 1998. Two lanes of concrete pavement were constructed in each test section: one on Union County H33, the other on Union County P33. The test section on H33 is located along the north and east city limits of Creston, Iowa, and will hereafter be referred to as the Urban test site. The other test section, is located on P33, south of US Highway 34 and the city of Creston, and will hereafter be referred to as the Rural test site. The location of both the Urban and Rural test sites is shown in Figure 3.1.

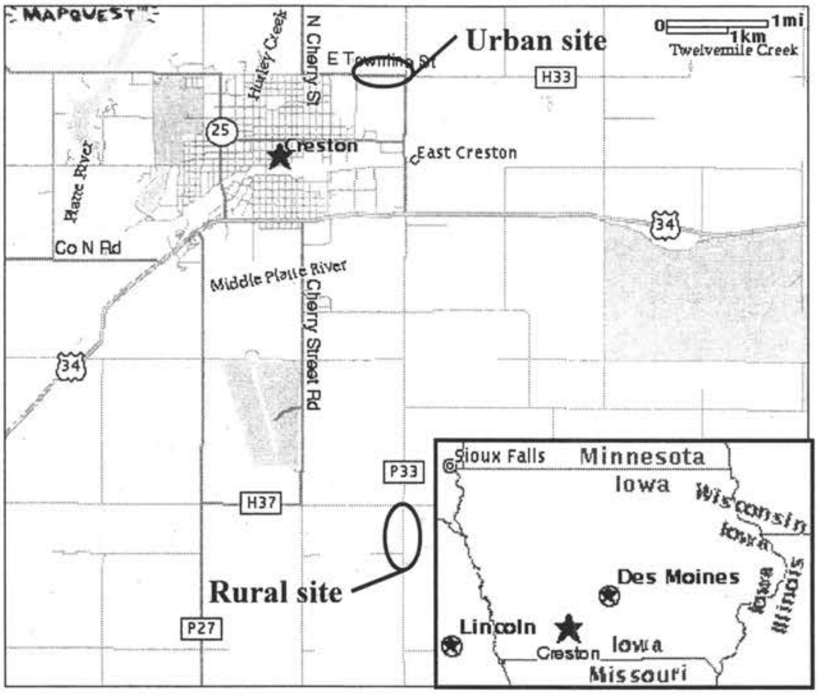


Figure 3.1 Map of site locations. [21]

3.2.1 Urban test site

The Urban test site is located on H33, west of the P33 and H33 intersection. The eastbound lane is within the Creston city limits and the westbound lane is under Union County jurisdiction. The Urban site consists of 802 feet of pavement divided into four different test sections. One test section contains no dowel bars across the westbound lane and a full basket in the eastbound lane. Another test section contains the standard dowel bar configuration (12-inch center-to-center spacing, offset 6 inches from the edge of pavement). The remaining two test sections consist of dowel bars placed only in the outside wheel path (12-inch center-to-center spacing, offset 6 inches from the edge of pavement), one section with three dowel bars, the other with four dowel bars.

It should be noted that due to concerns over the cost to repair a failed test section, the city of Creston elected not to build any sections with zero dowels bars in the eastbound lane. Therefore in the eastbound lane there were two sections that contained the standard dowel bar configuration. The location and number of dowel bars per joint per lane for each test section are tabulated in Table 3.1.

Table 3.1 Location and number of dowel bars for Urban test section.

Beginning station	End station	Number of dowels	Location	Number of joints
72+00	73+80	No Dowels[a]	-----	10
74+00	76+00	3 Dowels	Outside wheel path	11
76+20	78+00	4 Dowels	Outside wheel path	10
78+20	80+02	Full Basket	Full joint width	10

[a] Full basket in east bound lane

Typical dowel bar configurations for each test section are shown in Figure 3.2. The transverse joint is skewed (6:1) to ensure only one wheel load crosses the joint at a time.

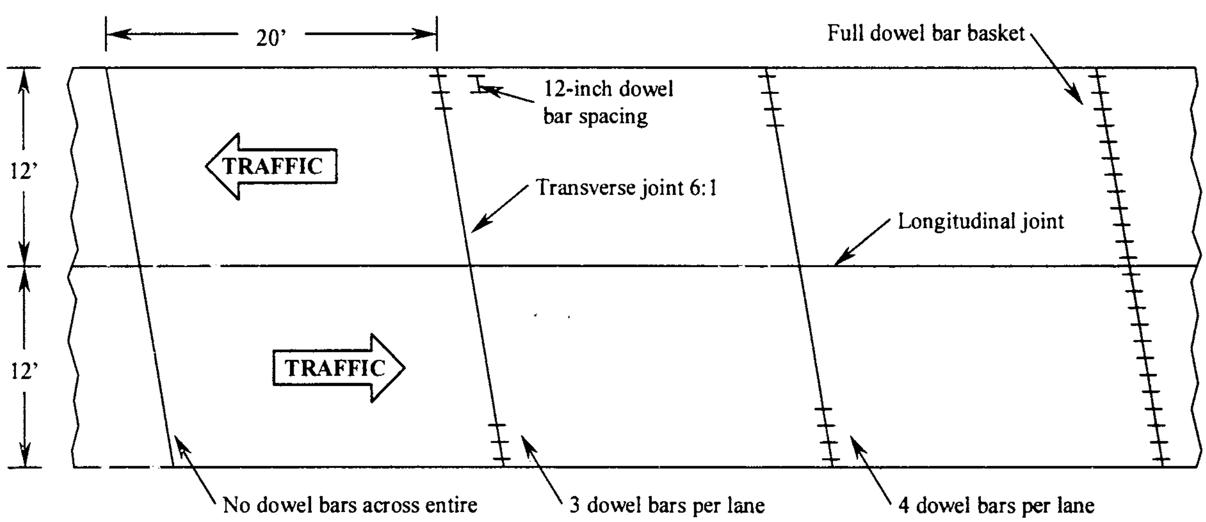


Figure 3.2 Typical dowel bar spacing configurations.

3.2.2 Rural test site

The Rural test site is part of a 6.4-mile project on P33, south of US Highway 34. The Rural test site consists of 1,596 feet of continuous pavement between stations 178+00 and 193+96. The Rural test site, which experiences lower volumes of traffic than the Urban test site (see section 4.2), is divided into four different test sections. One test section contains no dowel bars across the entire transverse joint; another section contains the standard dowel bar configuration across the entire transverse joint. The remaining two test sections consist of dowel bars placed only in the outside wheel path (12-inch center-to-center spacing, offset 6 inches from the edge of pavement), one section with three dowel bars, the other with four dowel bars.

Typical dowel bar configurations for each test section are shown in Figure 3.2 (section 3.2.1). The location and number of dowel bars per joint per lane for each test section are tabulated in Table 3.2.

Table 3.2 Location and number of dowel bars for Rural test section.

Beginning station	End station	Number of dowels	Location	Number of joints
178+00	181+80	No Dowels	------	20
182+00	185+80	3 Dowels	Outside wheel path	20
186+00	189+80	4 Dowels	Outside wheel path	20
190+10	193+96	Full Basket	Full joint width	20

3.3 Materials and Construction[2]

The Urban site consists of a ten-inch thick Portland cement concrete (PCC) over an existing bituminous pavement. The bituminous pavement, which exceeded eight inches in thickness throughout the entire project, was the result of a series of blade-laid cold mix layers

[2] The construction procedure was obtained from a report by Cable and Wosaba [22].

approximately 1.5 inches in depth. The first layer was placed in 1966 over a six inch rolled stone base. The bituminous surface was used "as constructed", i.e. it was not trimmed to a predetermined cross section.

The Rural site, in contrast, consists of a nine-inch thick PCC over a compacted soil base. The subgrade in the Rural site was trimmed to a uniform cross-section immediately prior to concrete placement. Subdrains were installed under the shoulder in both the Urban site and the Rural site to provide positive drainage of the subgrade. In order to verify that joints within the test area had formed (i.e. cracked completely through the thickness of the concrete at the joint) a visual survey was conducted immediately after construction.

3.3.1 Paving process

3.3.1.1 Urban site

The paving of the Urban test site was performed on August 31, 1998 by the Fred Carlson Co. The project was paved using a REX slipform paving machine, a full width-paving machine. The slipform paver was controlled by string lines on both sides of the roadway. The concrete was produced at a site ¼ mile north of US Highway 34 on P33 and delivered to the paving machine in dump trucks. The dump trucks carried the concrete to the site and deposited the material directly in front of the paving machine on the base material.

3.3.1.2 Rural site

The construction of the Rural project began on August 11, 1998 by the Fred Carlson Co., though, due to rainy conditions and other construction delays, the paving of the test site did not take place until August 16, 1998. The Rural test site was also paved using a REX slipform paving machine, but with an "Iowa Special" subgrade trimmer in front to form the proper subgrade cross section prior to concrete placement. Both the slipform paver and subgrade trimmer were controlled by string lines on each side of the roadway. The concrete was produced at a site ¼ mile north of US Highway 34 on P33 and delivered to the paving machine in dump trucks.

The dump trucks carrying the concrete to the site traveled over existing compacted subgrade to a location near the paving operation, where the trucks were backed up to the paving machine. The "Iowa Special" elevated the concrete from the dump truck, over the trimmer, and deposited the concrete in front of the slipform paving machine. This process minimized the impact of the loaded dump trucks on the subgrade while the trucks transported the load and backed up to the paving machine.

3.3.1.3 Longitudinal joint

Tie bars were placed across the longitudinal centerline joint in the pavement to tie adjoining lanes together. The tie bars ensure that the joint will remain tightly closed and that load will be transferred across the joint. The standard diameter of each tie bar is ½ inch with a length of 36 inches and a spacing of approximately 30 inches.

After the completion of the paving process, a longitudinal saw cut was made along the center of the pavement slab and skewed transverse joints were cut over the dowel basket assemblies. As concrete cures it shrinks, causing cracks to form. In order to control the location of shrinkage cracks, longitudinal and transverse joints are cut into the concrete. Both the longitudinal and transverse joints were sealed to reduce water infiltration into the subgrade.

3.3.2 Dowel assemblies

Prior to installation, the dowel basket assemblies were placed along both shoulders of the roadway. The intended location of the assemblies was painted on the subgrade at the edge of the paving area prior to the placement of the pavement. As the trimmer passed the intended assembly location the dowel basket assembly was moved into proper alignment. As previously stated, skewed joints were used in the pavement. Therefore, the dowel basket assemblies were also skewed (6:1) with the dowels aligned longitudinally in the pavement (see Figure 3.2). After the dowel baskets were moved into place, they were nailed into the subgrade (Figure 3.3)

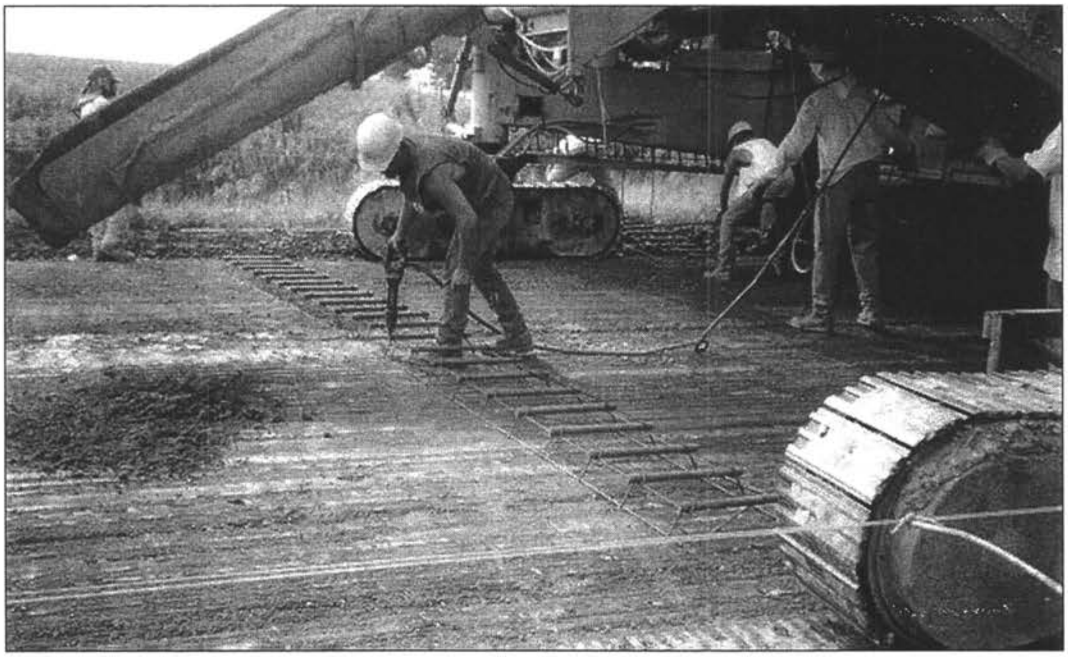


Figure 3.3 Installation of dowel basket assemblies (Rural site).

The installation of the partial basket assemblies (three or four dowels) followed the same procedure as that for the full basket assemblies (Figure 3.4). A transverse string line was used to align the smaller assemblies in the longitudinal direction. Figure 3.5 shows a cross-sectional view of a dowel basket assembly in pavement. It should be noted that it was more difficult to secure the basket assemblies to the bituminous base (Urban site) than to the compacted soil base (Rural site).

Figure 3.4 Placement of dowel basket assemblies (Urban site).

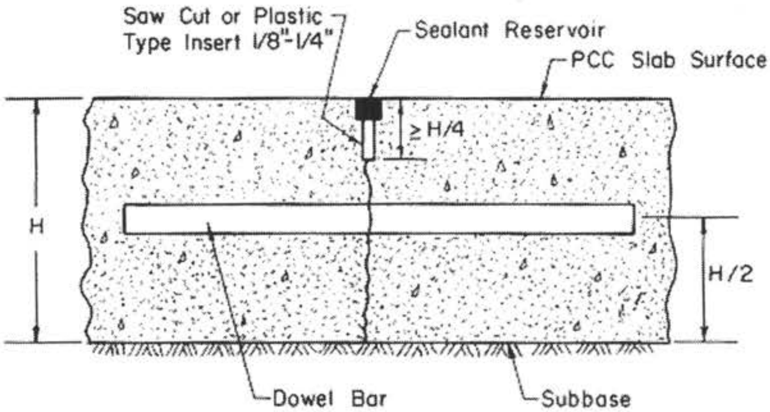


Figure 3.5 Dowel location in concrete pavement. [22]

3.3.3 Survey nail installation

During the placement of the concrete two survey nails (PK nails) were placed in the concrete on both sides of the roadway. The survey nails were inserted into the plastic concrete on each side of the joint (at a ten-inch nominal spacing, aligned longitudinally) to allow for future joint opening measurements. The location of the survey nails in the pavement is illustrated in Figure 3.6.

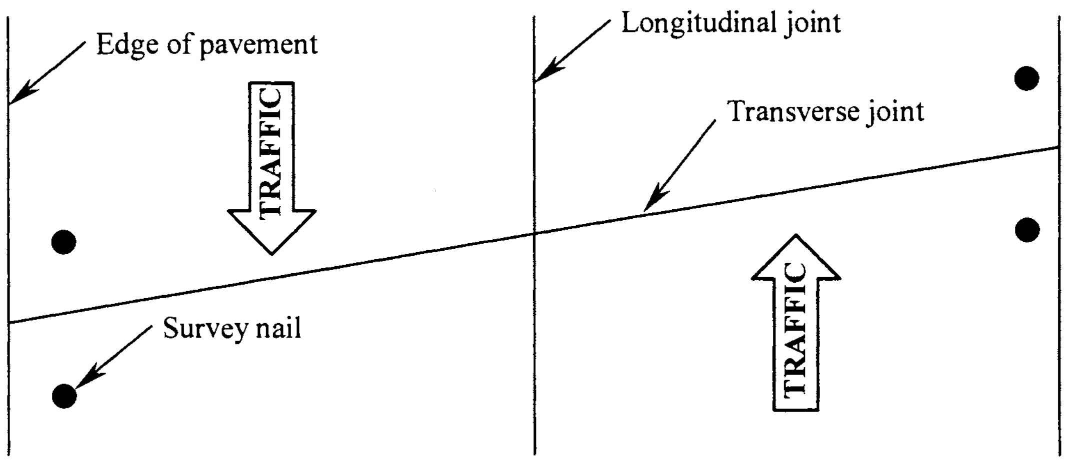


Figure 3.6 Diagram of survey nails in pavement.

3.4 Testing Program

As previously stated, the objective of this research project is to evaluate the impact of the number of dowel bars and dowel location on pavement performance and joint performance. In order to satisfy this objective, an evaluation of the test sections was performed biannually (early fall or late summer and early spring) over a five-year testing period. Testing in the spring would allow the evaluation of pavement with a typically wet, weak foundation, whereas in the early fall the subgrade is typically dry and strong. The evaluation of each test section in the Urban and Rural sites consisted of: 1) visual distress surveys, 2) joint opening, 3) joint faulting, 4) deflection measurements.

3.4.1 Visual distress

Visual distress surveys aid in identifying cracking, spalling, and changes in joint openings associated with changes in load transfer. The surveys identified distresses that occurred between testing periods. Surveys of the pavement were conducted in accordance with the SHRP (Strategic Highway Research Program) distress manual definitions [23]. ISU staff conducted visual distress surveys where distress types, extent, and severity were recorded for each survey period. The surveys were performed at the same time as the joint opening and faulting data were collected.

3.4.2 Joint opening

Concrete, like other materials, is affected by changes in temperature and changes in moisture content [24]. Concrete pavements will expand or shrink with changes in temperature according to their coefficient of thermal expansion. Concrete pavements will also expand or shrink with changes in moisture (e.g. shrinkage during curing). In order to avoid problems associated with thermal and moisture expansion and contraction, transverse joints are constructed to allow free horizontal movement. Therefore, joint openings were measured to ensure that the joints were allowed to move.

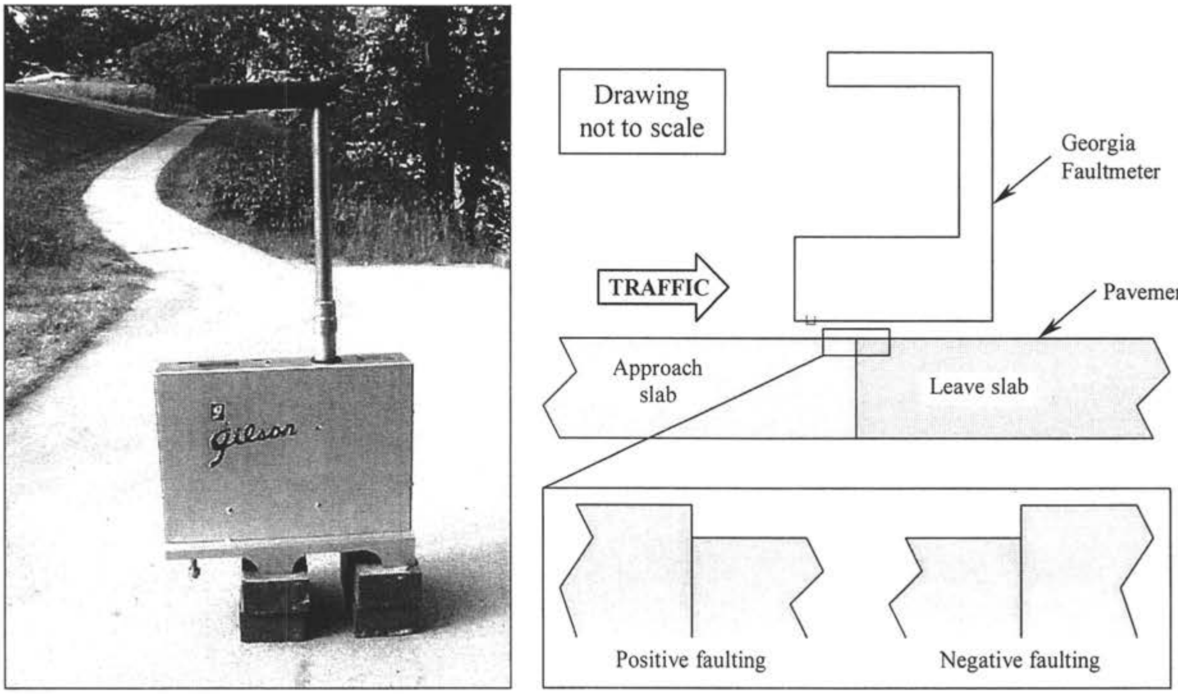
During construction of each test section, survey nails were placed at a ten-inch nominal spacing across each joint near the edge of the pavement (see section 3.3.3). Each joint opening was measured by using a digital caliper to measure the center-to-center distance of the survey nails. The joint openings were measured at the same time as the Road Rater testing.

3.4.3 Joint faulting

As pavement ages, it may begin to exhibit joint faulting, i.e. vertical displacement between slabs. Severe faulting creates an uneven driving surface and affects the quality of ride in a vehicle. Severe faulting can also cause pumping of the subgrade soils, which then creates voids below the pavement, reducing the strength of the pavement.

Joint faulting was measured using an electronic Georgia Digital Faultmeter with a digital readout that displays either positive or negative faulting. The faultmeter was set on the pavement in the direction of traffic and the faulting of the inside path and outside wheel path in each test section were measured.

A slab that is lower on the leave side of a joint will register as positive faulting on the faultmeter. A slab that is higher on the leave side of the joint will register as a negative faulting. A photo of the Georgia Faultmeter and a diagram of positive and negative faulting are illustrated in Figure 3.7.



(a (b

Figure 3.7 Georgia faultmeter and profile of positive and negative faulting.

3.4.4 Road Rater

The Road Rater is a trailer-mounted machine (Figure 3.8, Figure 3.9) that uses non-destructive testing methods to measure the response of a pavement section to a dynamic load similar in magnitude to that produced by a moving vehicle tire load. A load is hydraulically lowered to the pavement and oscillated to produce a loading force [17].

The loading force is determined by the following equation:

$$F = 32.70 * f^2 D \qquad \textit{Equation 1.1}$$

where F = the peak-to-peak force (lb)
f = loading frequency (Hz)
D = peak-to-peak mass displacement (inch)

A force setting of 30 Hz and mass displacement of 0.068 inches are recommended for a rigid pavement, which produces a peak-to-peak force of 2,000 pounds [17].

Figure 3.8 Photograph of a Road Rater and towing vehicle.

Figure 3.9 Photograph of a Road Rater.

The deflection of the pavement due to the applied load is measured by four velocity sensors and collected by a computer in the tow vehicle. One velocity sensor is positioned directly beneath the load; the other three sensors are positioned at one-foot intervals. The deflection of the pavement is illustrated in Figure 3.10 and the Road Rater schematic is illustrated in Figure 3.11.

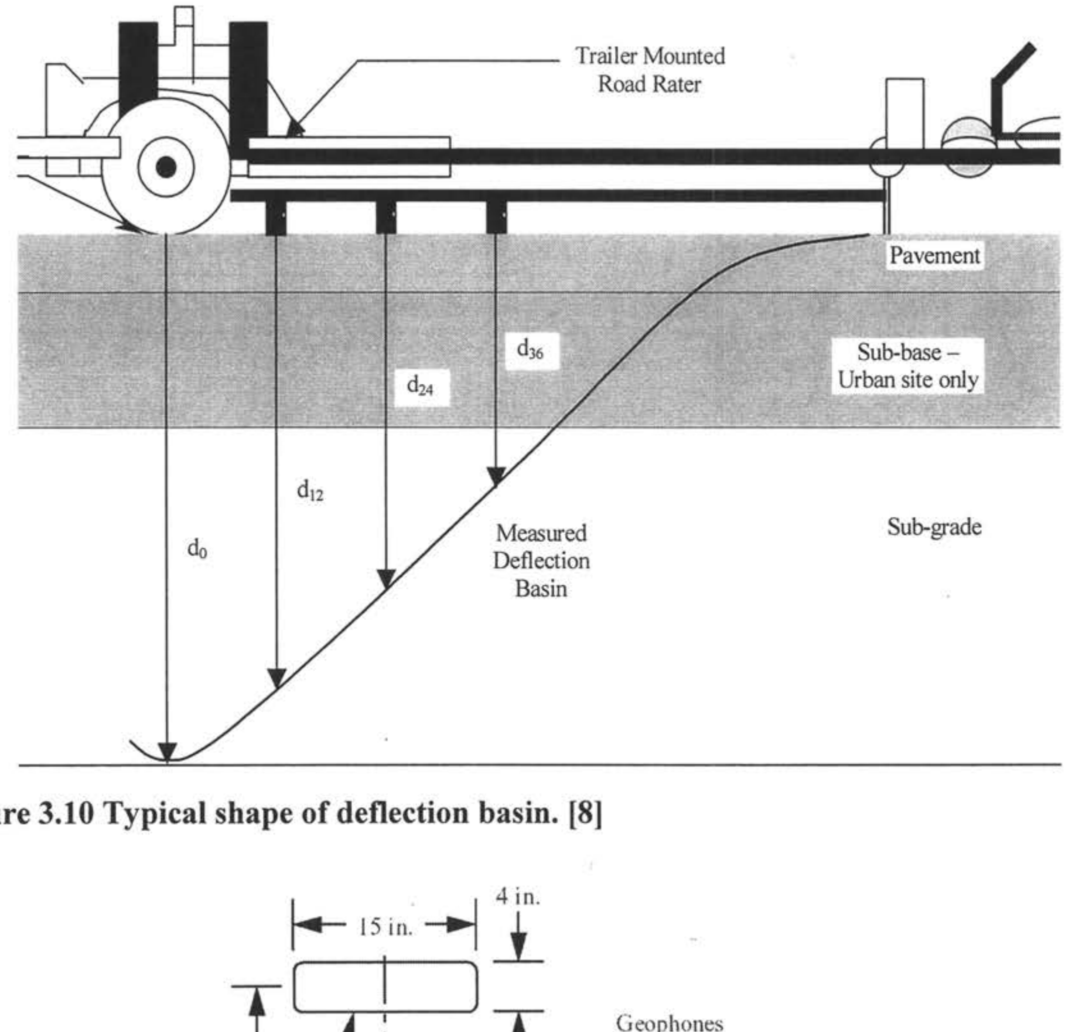


Figure 3.10 Typical shape of deflection basin. [8]

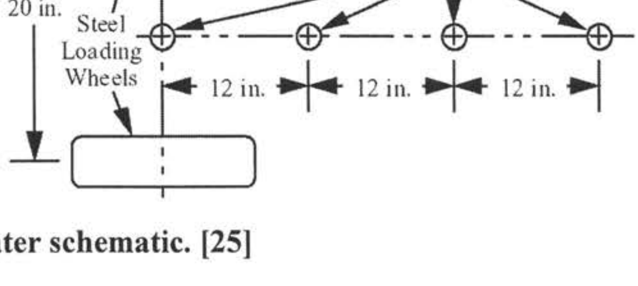


Figure 3.11 Road Rater schematic. [25]

The deflection of the pavement at the joint provides information on the load transfer efficiency. In contrast, the deflection at the midpanel (i.e. near the center of the slab) is a function of the pavement and the subgrade stiffness. The deflection information obtained by the Road Rater is then analyzed to provide an indication of the performance of the pavement.

A Road Rater was used by the Iowa DOT to perform deflection testing in the inside and outside wheel paths for both lanes of traffic in both the Urban site and the Rural site. The Road Rater deflection tests were conducted two feet from the edge of pavement in the outside wheel path, and one foot from the centerline of the pavement for the inside wheel path. The deflection data from the Road Rater was used to determine variances in joint deflections and load transfer across transverse joints. Information provided by the Road Rater was also used in the backcalculation of layer moduli.

4. DATA ANALYSIS

4.1 Soil Survey

As part of the evaluation of this research project a soil identification was performed for both sites. The soil identification included in-situ soil classification from soil borings conducted by the Iowa DOT and consultation of US Department of Agriculture (USDA) soil survey. The soil borings were conducted on the shoulder of the roadway (16 or 17 feet from the centerline). The boring logs for the Urban and Rural test sites can be found in Appendix A: Soil Boring Logs. The summary of the findings from the soil borings and the USDA soil survey are summarized below.

4.1.1 Urban Site

4.1.1.1 Soil survey [26]

The predetermined soil series at the Urban site is Winterset silty clay loam (369). Winterset silty clay loam is located on nearly level broad upland ridge tops. The Winterset series consists of poorly drained soils that were formed in loess under native vegetation of prairie grasses. The series is classified under AASHTO (American Association of State Highway and Transportation Officials) as an A-7, and under the Unified Soil Classification System (USCS), as a CL, OL, and/or CH. The location of the Urban site with respect to the soil survey map is illustrated in Figure 4.1.

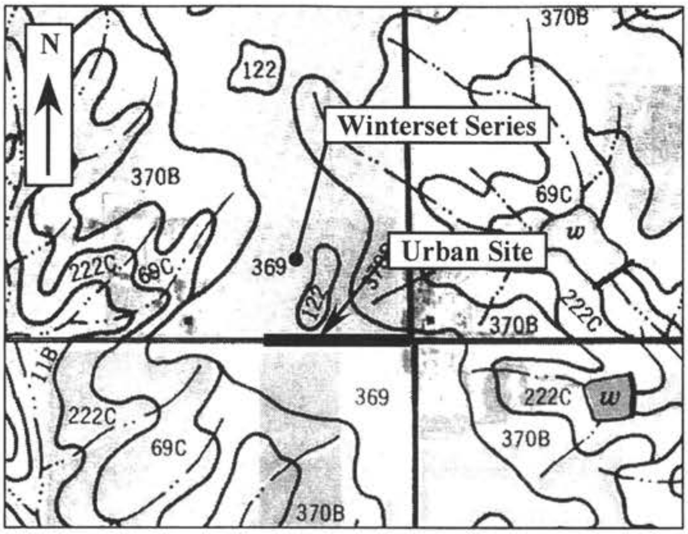


Figure 4.1 Soil survey for Urban site. [26]

A representative subsurface profile for Winterset series contains:

- A surface layer of black silty clay loam approximately 18 inches thick
- A subsurface layer to a depth of approximately 52 inches, with
    - An upper section of very dark grey, heavy, silty clay loam
    - A middle section of dark grayish brown, heavy, silty clay loam
    - A lower section of grayish brown, silty clay loam
- A substratum of mottled grayish brown and light olive brown light silty clay loam

The expected liquid limit (LL) and plasticity index (PI) ranges for the top 18 inches of the soil profile are 40-50 and 20-30, respectively. The LL and PI ranges for 18-42 inches in depth are 50-70 and 30-40, respectively. For a depth of 42-75, the LL and PI ranges are 40-50 and 25-35, respectively.

4.1.1.2 Boring log summary

In the top foot of soil, the borings encountered "road metal", i.e. shoulder stone and shoulder material. The road metal was underlain by varying thickness (from two to ten feet) of fill material (black to grey silty clay). Beneath the fill, the borings encountered grey silty clay (identified as loess), which was underlain by grey clay (identified as weathered glacial till). The results from the soil borings correlated with the expected conditions identified in the USDA soil survey.

4.1.2 Rural Site

4.1.2.1 Soil survey [26]

There are three different soil series in the Rural site. The soil series located at the north end of the site is Nira-Sharpsburg silty clay loam (317C). Two areas of the site contain Clearfield silty clay loam (69C), and the remainder of the site is Sharpsburg silty clay loam (370B). The location of the Rural site with respect to the soil survey map is shown in Figure 4.2.

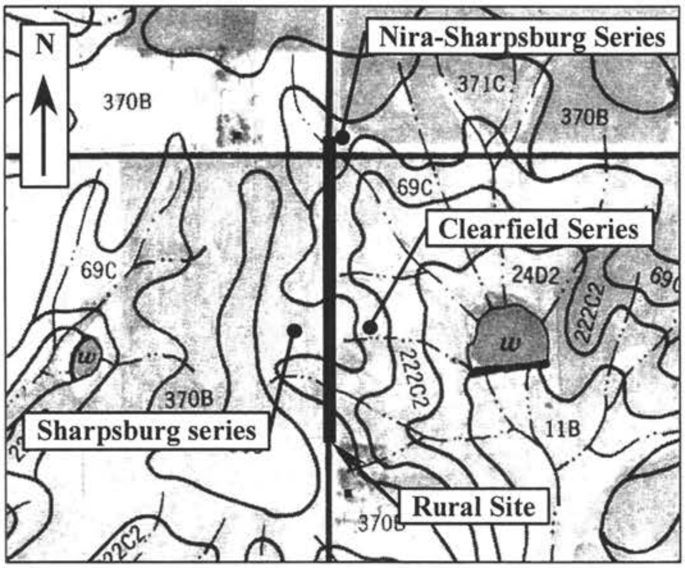


Figure 4.2 Soil survey for Rural site. [26]

The Nira-Sharpsburg (60% Nira series, 35% Sharpsburg series) silty clay loam is located on side slopes surrounding nearly level, stable upland divides. Nira-Sharpsburg series consist of moderately well drained soils formed in loess under native vegetation of prairie grasses. The series is classified under AASHTO as A-6 or A-7, and under the USCS, as a CL, OL, CH, and/or OH.

A representative subsurface profile for Nira series contains (for the Sharpsburg series profile see section profile, below):

- A surface layer of black and very dark grey silty clay loam approximately 13 inches thick
- A subsurface layer to a depth of approximately 44 inches, with:
    - An upper section of brown, silty clay loam
    - A lower section of grayish brown, silty clay loam
- A substratum of mottled grayish brown and brown, silty clay loam

The expected LL and PI ranges for the top 13 inches of the Nira soil profile are 41-50 and 15-25, respectively. The LL and PI ranges for 13-30 inches in depth are 41-50 and 20-30, respectively. For a depth of 30-60, the LL and PI ranges are 35-45 and 15-25, respectively.

The Sharpsburg series is located on gently sloping to strongly sloping divides and side slopes of uplands. The Sharpsburg series consist of moderately well drained soils formed in loess under native vegetation of prairie grasses. The series is classified under AASHTO as A-6 or A-7, and under the USCS, as a CL, OL, CH, and/or OH.

A representative subsurface profile for Sharpsburg series contains:

- A surface layer of black, very dark brown, and very dark grayish brown silty clay loam approximately 13 inches thick
- A subsurface layer to a depth of approximately 50 inches, with:
    - An upper section of brown, silty clay loam
    - A middle section of mottled brown and dark yellowish brown, silty clay loam
    - A lower section of grayish brown and dark yellowish brown, silty clay loam
- A substratum of mottled grayish brown and yellowish brown, silty clay loam

The expected LL and PI ranges for the top 13 inches of the Sharpsburg soil profile are 35-55 and 20-30, respectively. The LL and PI ranges for 13-40 inches in depth are 35-60 and 20-35, respectively. For a depth of 40-62, the LL and PI ranges are 35-50 and 20-30, respectively.

The Clearfield silty clay loam is located on moderately sloping slopes on sideslopes and coves on uplands. The Clearfield series consist of poorly drained soils formed in three to six feet of loess over clayey glacial till, under native vegetation of prairie grasses. The series is classified under AASHTO as A-7, and under the USCS, as a CL, OL, CH, OH, and/or MH.

A representative subsurface profile for Clearfield series contains:

- A surface layer of black and very dark grey friable, silty clay loam approximately 17 inches thick
- A subsurface layer to a depth of approximately 48 inches with:
    - An upper section of dark grey, firm, silty clay loam
    - A middle section of olive grey, firm, silty clay loam
    - A lower section of light brownish grey to grey, friable, silty clay loam
- A substratum of dark grey to black, firm, heavy, silty clay loam grading to silty clay

The expected LL and PI ranges for the top 13 inches of the soil profile are 45-55 and 20-30, respectively. The LL and PI ranges for 13-48 inches in depth are 50-60 and 25-35, respectively. For a depth of 48-60, the LL and PI ranges are 55-70 and 35-45, respectively.

4.1.2.2 Boring log summary

In the top foot of soil, the borings encountered road metal. The road metal was underlain by varying thickness (from one to five feet) of fill material (dark brown silty clay). Beneath the fill, the borings encountered black and/or grey brown silty clay (identified as loess). The results from the soil borings correlated with the expected conditions identified in the USDA soil survey.

4.2 Traffic Data

The volume of traffic experienced by a pavement impacts pavement's performance through fatigue. In order to estimate the volume of traffic in the Urban and Rural sites, traffic counts performed by the Iowa DOT were consulted [27]. Traffic counts were measured in AADT (annual average daily traffic), which estimates annual average daily traffic based on 24-hour counts taken over an entire year. The traffic counts obtained for 1996 and 2000 are tabulated in Table 4.2

Table 4.1 Annual average daily traffic (in vehicles).

	Year	AADT
Urban	1996	1270
Rural	1996	90
	2000	140

4.3 Visual Distress Survey

The visual surveys conducted by ISU staff identified few distresses. In the Urban site two corner cracks were detected, one of which appeared to be caused by a motorgrader during shoulder construction. The Rural site, in contrast, contained 13 small corner cracks (the largest being three inches by eight inches in size). One transverse crack at midslab and one small spall were also detected in the Rural site. The midslab crack is located at an intersection and mirrors the centerline joint of the approach slabs. The crack was caused by the construction method of the intersection, which included slab tying and joint development across the intersection.

4.4 Joint Opening

The joint opening data was collected twice a year at approximately the same time each fall and spring. As previously stated, concrete expands and contracts with changes in moisture and temperature. Therefore it is expected that the dowel bar arrangement does not have a significant effect on the joint opening. The change in average joint opening (between current and previous survey date) changed over time, implying the pavement slabs were allowed to expand and contract. Slightly larger joint opening changes occurred in the Rural site as compared to the Urban site. The average joint opening during the research time for Urban and Rural sites is tabulated in Table 4.2. Tabulated results and graphical representations of the joint opening field evaluation can be found in Appendix B.

Table 4.2 Change in average joint opening during research lifetime (in 1/1000 inch).

	Urban	Rural
Zero dowel bars	-3.54	-7.87
Three dowel bars	-0.49	18.61
Four dowel bars	-2.51	10.47
Full dowel bars	8.49	32.47

4.5 Statistical Analyses

The statistical analyses for this research project were challenging due to the complex interaction of the pavement system with its environment and vehicle traffic. The independent variables that were studied are: dowel bar arrangement (location and number), location within the lane (inside wheel path or outside wheel path), lane direction (east or west for Urban site and north or south for Rural site), and time period (entire testing period, fall, or spring). The dependent variables that were investigated are: joint faulting (inside wheel path or outside wheel path), load transfer, soil modulus (k), and area. The dependent and independent variables are associated as follows: the dependent variables are affected by the independent variables.

ANOVA (analysis of variance) analyses were performed using SAS® (Statistical Analysis System) to determine which independent variables had a significant effect on each dependent variable. ANOVA analyses are used to test for a significant difference between means (averages) of a specific dependent variable taken from different samples. The difference in means is deemed significant (implying that the independent variable has an effect on the dependent variable) when a significance level, specified by the researcher, is reached. The significance level for this research project is 0.05, i.e. it is acceptable that 5% of the relationships deemed significant are in fact not significant. The ANOVA analyses do not determine which dowel arrangement is the best, they only determine if there is a significant difference between the dowel placements. The following sections will discuss the results of the field evaluations and summarize the results of the statistical analyses.

4.6 Joint Faulting

The joint faulting data was also collected twice a year at approximately the same time each fall and spring. As previously stated, a pavement may begin to exhibit joint faulting as it ages due to the decreasing ability to transfer load by aggregate interlock. Therefore it is expected that the dowel bar arrangement will have an effect on the faulting of the pavement. Additionally, it is expected that the pavement with no dowel bars will have the most faulting across the width of the pavement.

ANOVA analyses were performed to determine the significance between dowel arrangement and joint faulting. The statistical results are tabulated in Table 4.3. The average joint faulting for Urban and Rural sites is tabulated in Table 4.4 and represented graphically in Figure 4.3. Additional tabulated results and graphical representations of the joint faulting can be found in Appendix B.

Table 4.3 Average joint faulting statistical results.

	Sample size	Average faulting (1/1000 inches)	Significance for dowel number	Statistically significant?
Urban	492			
Inside wheel path		-0.42	<0.0001	yes
Between 0 and full dowels			0.152	no
Between 3 and full dowels			0.000	yes
Between 4 and full dowels			0.000	yes
Outside wheel path		-0.54	0.3547	no
Between 0 and full dowels			0.530	no
Between 3 and full dowels			0.210	no
Between 4 and full dowels			0.158	no
Rural	1120			
Inside wheel path		-0.25	0.4419	no
Between 0 and full dowels			0.349	no
Between 3 and full dowels			0.536	no
Between 4 and full dowels			0.465	no
Outside wheel path		-0.25	0.0024	yes
Between 0 and full dowels			0.082	no
Between 3 and full dowels			0.000	yes
Between 4 and full dowels			0.000	yes

The test results indicate:

- In the Urban site, the arrangement (number and location) of dowel bars has a significant effect on joint faulting for the inside wheel path, but not for the outside wheel path.
- The difference in joint faulting between zero and full dowels for the Urban site is not significant.

- The joint faulting for the fully dowel section in the Urban site is significantly higher than the joint faulting for the three or four dowel sections.
- In the Rural site, the joint faulting was significantly affected by the dowel arrangement in the outside wheel path, but not for the inside wheel path.
- The difference in joint faulting between full dowels and the alternative dowel arrangements is not significant for the inside wheel path in the Rural site.
- The difference in joint faulting between zero dowels and full dowels is not significant for the outside wheel path in the Rural site.
- The joint faulting for the full dowels in the outside wheel path in the Rural site is significantly higher than the joint faulting for three or four dowels.
- With the exception of the full dowel bar section in the Urban site, the zero dowel bars sections had the largest faulting.
- There appeared to be no trend with the remaining dowel bar placements.

Table 4.4 Average joint faulting for research lifetime (in 1/1000 inch).

	Urban		Rural	
	Inside wheel path	Outside wheel path	Inside wheel path	Outside wheel path
Zero dowel bars	-25.08	-22.88	-20.85	-16.82
Three dowel bars	-3.93	-18.45	-17.63	-5.23
Four dowel bars	-4.19	-19.11	-4.63	-10.27
Full dowel bars	-32.99	-29.18	3.82	-11.38

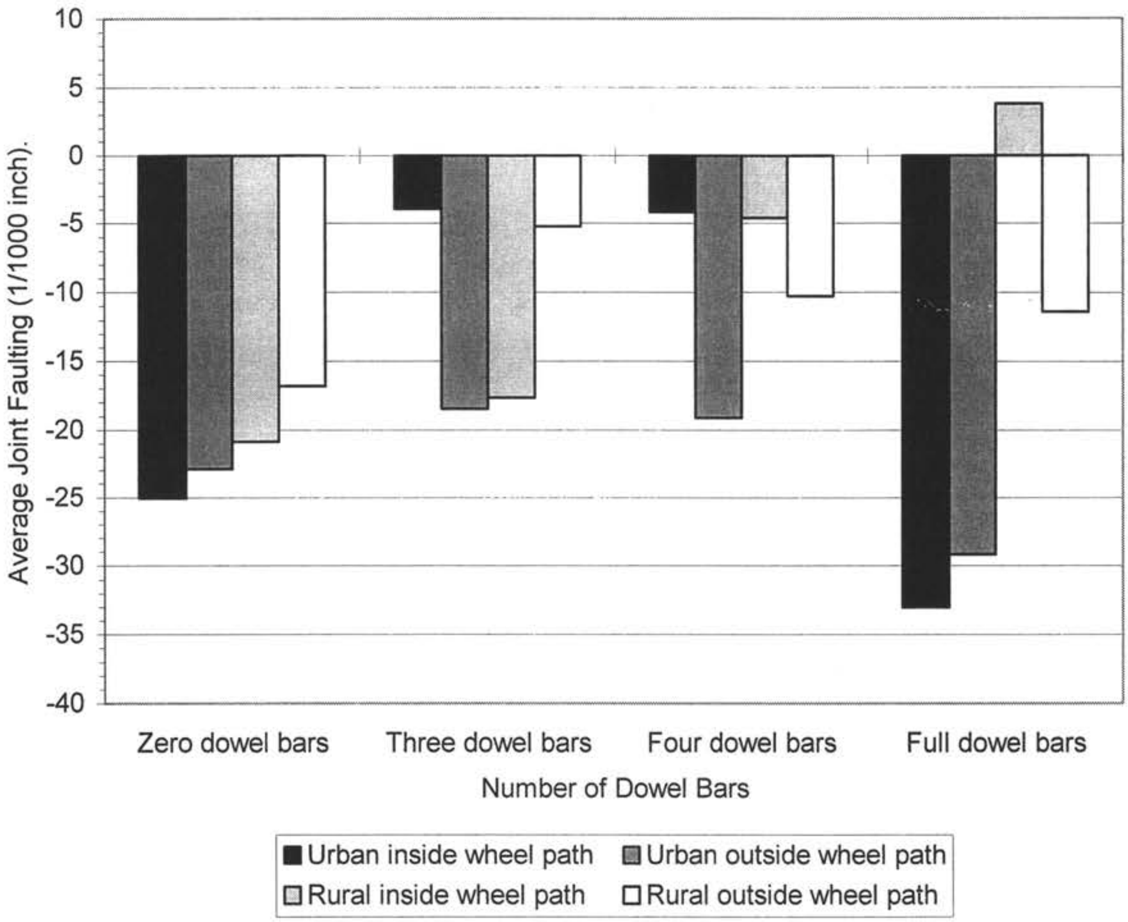


Figure 4.3 Average joint faulting for research lifetime vs. number of dowel bars.

A large variability in joint faulting may have been introduced by the method of measurement. Joint faulting measurements are affected to a large extent by the location at which the faultmeter is placed. During the finishing of the pavement, the surface is tined (i.e. small grooves are scratched into the surface of the pavement). If the plunger on the faultmeter (Figure 3.7, section 3.43) was positioned between two tined grooves, a smaller faulting would be recorded than if the plunger was positioned in a tined groove. Therefore, additional care is required in the measurement of joint faulting if the measurements are to be used for performance measurements.

4.7 Deflection Data

As previously stated, a Road Rater was used by the Iowa DOT to perform deflection testing for this research project. Deflection data was collected twice a year at approximately the same time each fall and spring. The deflection measurements (in mils, i.e. 1/1000 inch) were recorded for the Urban and Rural sites in the inside and outside wheel paths in both lanes of traffic. The Road Rater contains four velocity sensors (Figure 4.4), at a 12-inch spacing, to measure the deflection of the pavement.

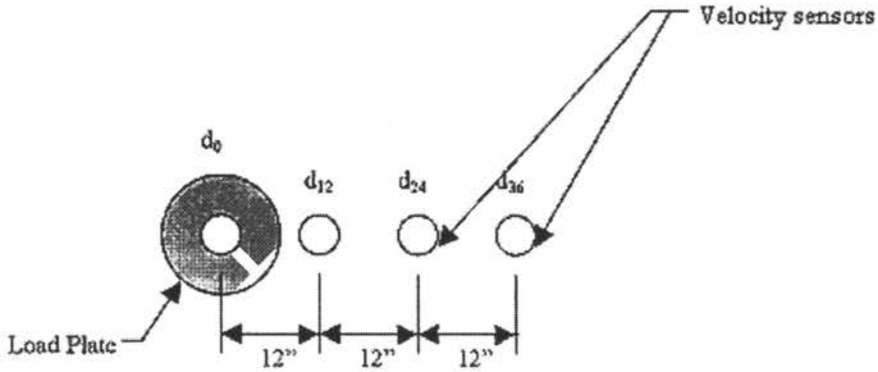


Figure 4.4 Arrangement of Road Rater velocity sensors. [8]

4.7.1 Midpanel deflection

4.7.1.1 Calculations

Deflections measured at the midpanel (i.e. near the center of the slab) are analyzed to determine the cross-sectional area of the deflection basin and the modulus of subgrade reaction. The modulus of subgrade stiffness is determined through a backcalculation procedure based on Westergaard's plate theory model [28]. The deflection basin area is calculated using Equation 4.1, which is derived from the trapezoidal rule [28]. The deflection basin area is calculated using raw data; therefore, the area parameter must be normalized by dividing the deflection at each sensor by the maximum deflection. The normalization of the area parameter removes variations in the deflections due to the applied load.

$$\text{Area} = 6 \times \left(1 + 2\frac{\text{Sensor 2}}{\text{Sensor 1}} + 2\frac{\text{Sensor 3}}{\text{Sensor 1}} + \frac{\text{Sensor 4}}{\text{Sensor 1}}\right) \qquad \textit{Equation 4.1}$$

where: Sensor 1 = Deflection at sensor located directly beneath the applied load
Sensors 2, 3, and 4 = Deflection at sensors located at 12, 24, and 36 inches from the applied load, respectively

The area parameter is used to estimate a radius of relative stiffness, ℓ, which is then used to calculate the modulus of subgrade reaction, k. The radius of relative stiffness is defined as the ratio of the stiffness of the slab or pavement to the stiffness of the foundation soils [28]. The radius of relative stiffness is calculated using the following equation, which assumes a dense liquid foundation:

$$\ell = \sqrt[4]{\frac{E * h^3}{12 * (1 - \mu^2) * k}} \qquad \textit{Equation 4.2}$$

where: ℓ = Radius of relative stiffness (inch)
E = Concrete modulus of elasticity (psi)
h = Concrete thickness (in)
μ = Poisson ratio of concrete, typically between 0.10 and 0.20 for concrete [24]
k = Modulus of subgrade reaction

Hall developed an equation (Equation 4.3) for the radius of relative stiffness as a function of the deflection basin area [28].

$$\ell = \left[\frac{\ln\left(\frac{36 - area}{1812.279}\right)}{-2.559}\right]^{4.387} \qquad \textit{Equation 4.3}$$

As previously stated, the radius of relative stiffness is used to calculate the modulus of subgrade reaction, which is a measure of the stiffness of the subgrade. The modulus of subgrade reaction is calculated from the measured deflections using the following equation [29]:

$$k = \frac{d_i * P}{D_i * \ell^2} \qquad \textit{Equation 4.4}$$

where: k = Modulus of subgrade reaction
d_i = Non-dimensional sensor deflection corresponding to measured deflection , D_i
P = Applied load (lb)
D_i = Deflection at a distance i from the load plate
ℓ = Radius of relative stiffness (inch)

Equation 4.4 uses only one deflection measurement to determine the subgrade reaction; therefore an average modulus of subgrade reaction can be determined from the four sensor readings.

The modulus of subgrade reaction determined by the Road Rater, is a dynamic modulus. The value of a dynamic modulus is approximately twice that of the corresponding static modulus, which is used in pavement design. Therefore, the modulus of subgrade reactions determined from this research project should be halved if they are to be used in a future roadway design (e.g. widening of the roadway). It should be noted here that typical modulus values refer to a static modulus of subgrade reaction; therefore the results of this research project should also be divided in half to be compared with typical values.

4.7.1.2 Deflection basin area

The average deflection basin area is a good indicator of the structural soundness of a pavement over time. A stable pavement tends to have an average area of 30-32 inches [30]. An ANOVA analysis was performed to determine the significance between dowel arrangement and the deflection basin area. From the statistical results (tabulated in Table 4.5), it was determined that the arrangement (number and location) of dowel bars does not have a significant effect on the deflection basin area. A graphical analysis showed that there was no trend between the dowel arrangement and area parameter.

Statistical and graphical analyses indicate dowel arrangement does not affect the area parameter; therefore there will be no further discussion of the area parameter. Tabulated results and graphical representations of the deflection basin area can be found in Appendix D.

Table 4.5 Significance levels for average deflection basin area during research lifetime.

	Sample size	Average area (inch)	Significance for dowel number	Statistically significant?
Urban	1141	31.23	0.1988	no
Rural	2053	30.44	0.1019	no

4.7.1.3 Dynamic modulus of subgrade reaction

ANOVA analyses were performed to determine the significance between the dynamic modulus of subgrade reaction and season (spring or fall), location within the lane (inside or outside wheel path), and dowel arrangement. Additional ANOVA analyses were conducted to compare the effect of the full compliment of dowel bars with the other dowel placements. The statistical results are tabulated in Table 4.6. The average dynamic modulus of subgrade reaction for the Urban and Rural sites is tabulated in Table 4.7 and represented graphically in Figure 4.5 through Figure 4.9. Additional tabulated results and graphical representations of the dynamic modulus of subgrade reaction can be found in Appendix E.

Table 4.6 Significance levels for average dynamic k-value during research lifetime.

	Sample size	Average dynamic k-value, (pci)	Significance	Statistically significant?
Urban	1141	212.96		
Season			<0.0001	yes
Location within the lane			<0.0001	yes
Dowel arrangement			<0.0001	yes
0 and full			<0.0001	yes
3 and full			<0.0001	yes
4 and full			<0.0001	yes
Rural	2053	182.07		
Season			<0.0001	yes
Location within the lane			<0.0001	yes
Dowel arrangement			<0.0001	yes
0 and full			0.0002	yes
3 and full			0.6352	no
4 and full			0.2567	no

The test results indicate:

- The season (spring or fall) has a significant effect on the dynamic modulus of subgrade reaction.
- The location within the lane (inside or outside wheel path) of dowel bars has a significant effect on the dynamic modulus of subgrade reaction.
- The arrangement (number and location) of dowel bars has a significant effect on the dynamic modulus of subgrade reaction.
- There is a significant difference between the effect of the full dowel compliment and each of the other dowel placements on the dynamic modulus of subgrade reaction, for the Urban site.
- There is a significant difference between the effect of the full dowel compliment and zero dowels on the dynamic modulus of subgrade reaction for the Rural site.
- There is not a significant difference between the effect of the full dowel compliment and three or four dowels on the dynamic modulus of subgrade reaction for the Rural site.

Table 4.7 Average dynamic modulus of subgrade reaction for research lifetime.

	Urban		Rural	
	Inside wheel path	Outside wheel path	Inside wheel path	Outside wheel path
Zero dowel bars	218.31	201.02	214.19	153.50
Three dowel bars	222.30	206.06	210.03	136.70
Four dowel bars	221.07	205.51	212.94	136.68
Full dowel bars	221.45	207.99	214.80	177.71

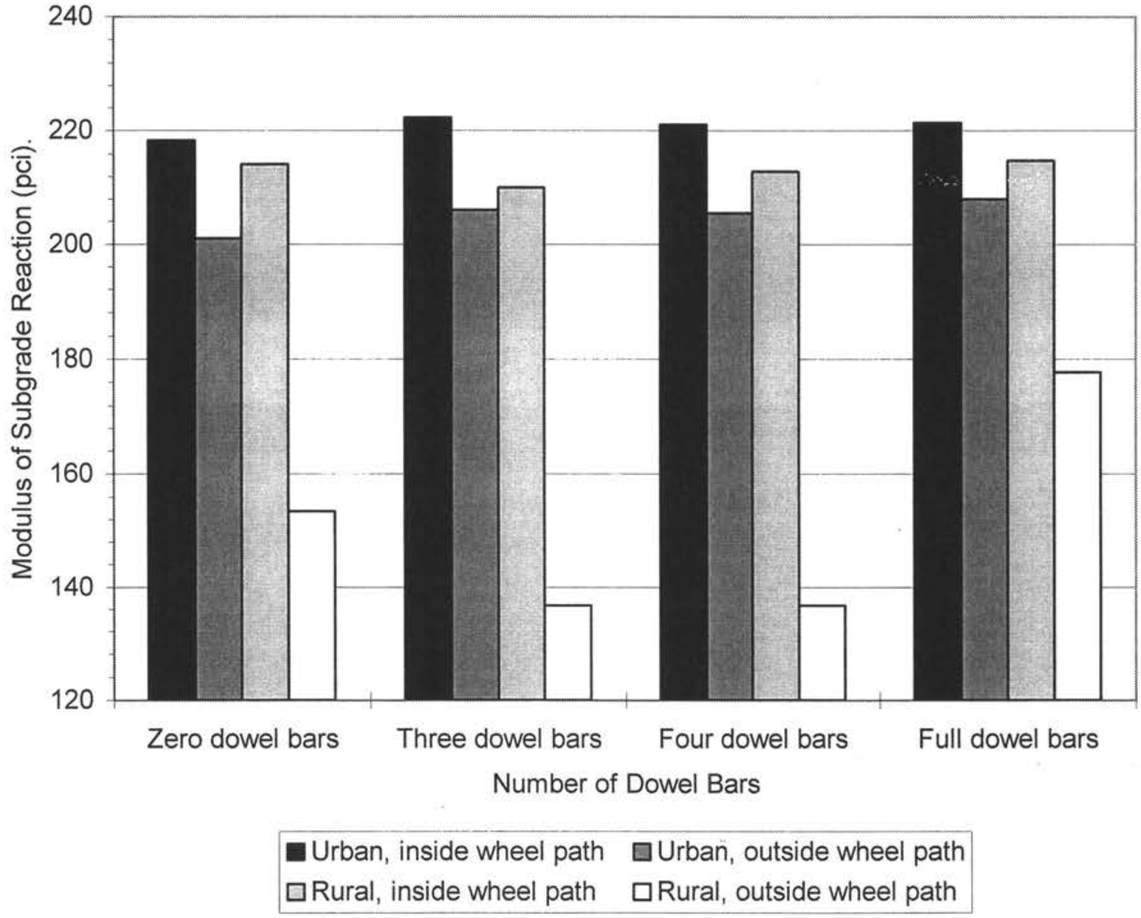


Figure 4.5 Average research lifetime k-value vs. number of dowel bars.

Figure 4.5 compares the average dynamic modulus of subgrade reaction (dynamic k-value) during the research period for each dowel arrangement. The dynamic k-value for the Urban site, in both the inside and outside wheel paths, appears to be consistent. The dynamic k-value for the Rural site, in contrast, is not consistent in both wheel paths. The outside wheel path in the Rural site displays the lowest average dynamic k-value and displays the largest variance between dowel placements. The difference in dynamic k-value between the Urban and Rural sites may be caused by the difference in subgrade support. The Urban site has an asphalt subgrade, which will increases the stability and strength of the pavement system. The subgrade for the Rural site, in contrast, is compacted soil, which may degrade over time due to changes in subgrade moisture, freeze-thaw conditions, and other changes in subgrade conditions.

The difference in dynamic k-value between wheel paths in the Rural site may be caused by infiltration of water and untied shoulders. The subgrade under the inside wheel path in the Rural site is partially protected by the pavement, whereas the subgrade beneath the edge of the pavement is less protected from external effects (such as infiltration of rainwater). Infiltration of water at the edge of the pavement could cause saturated soil conditions due to poorly draining soils (see section 4.1.2). Saturated soil conditions could lead to pumping of the subgrade, which could further decrease the strength of the subgrade.

Also, compacted granular shoulders were used in lieu of tied concrete shoulders. Tied concrete shoulders reduce edge and corner deflections in addition to reducing water infiltration [7]. The use of compacted granular shoulders did not reduce edge and corner deflection, thereby increasing the fatigue in the pavement and weakening the subgrade.

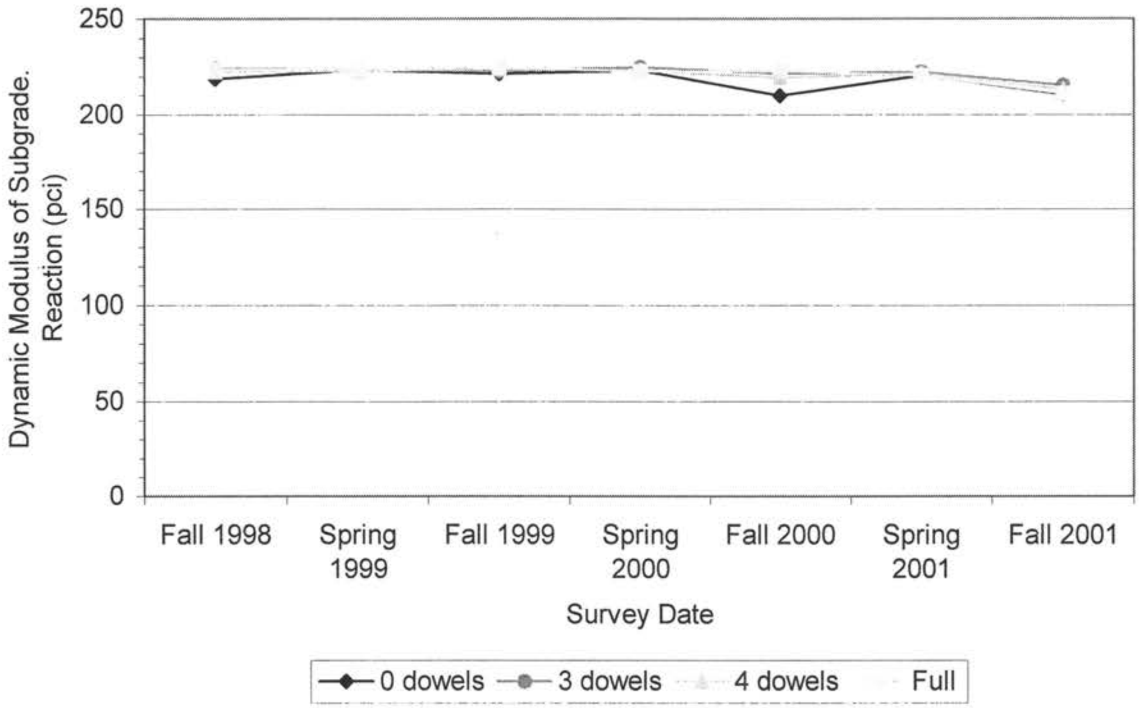


Figure 4.6 Average dynamic k-value vs. survey date for Urban site, inside wheel path.

Figure 4.6 compares the dynamic k-values for each dowel arrangement over time in the Urban site for the inside wheel path. There appears to be little difference in the dynamic k-values over time for each of the dowel bar placements with the exception of zero dowel bars in Fall 2000. There was also little difference between each dowel arrangement for any given survey date, again with the exception of the zero dowel bars in Fall 2000.

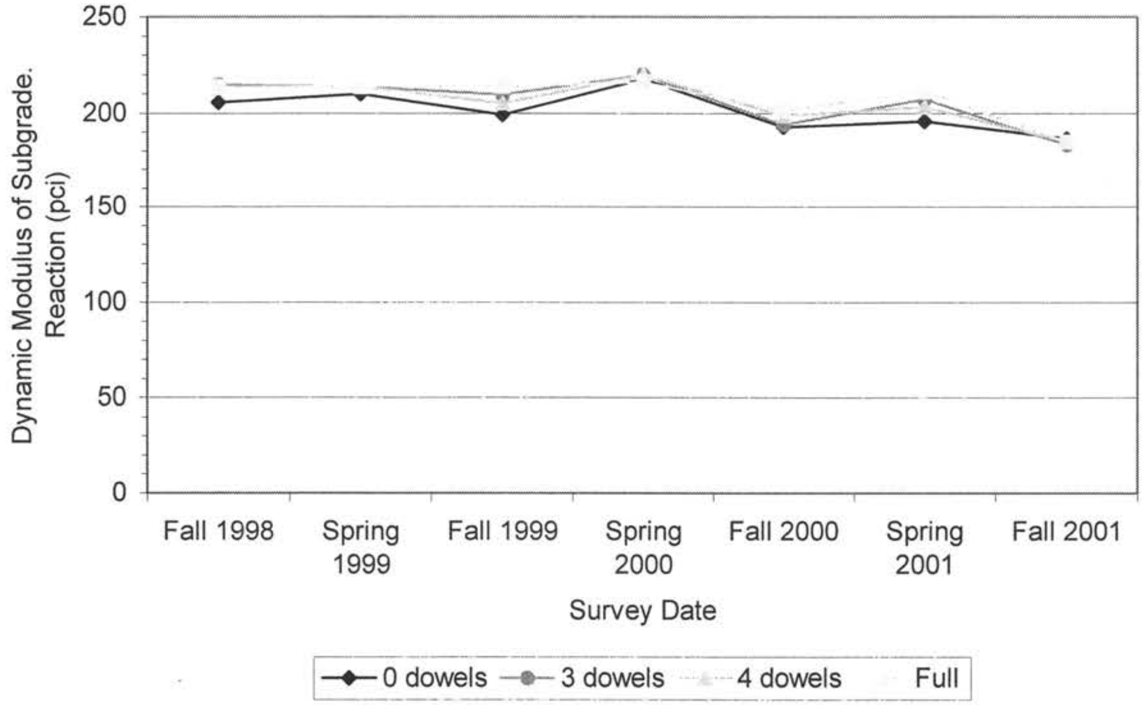


Figure 4.7 Average dynamic k-value vs. survey date for Urban site, outside wheel path.

Figure 4.7 compares the dynamic k-values for each dowel arrangement over time in the Urban site for outside wheel path. Similar to the inside wheel path, there appears to be little difference in the dynamic k-value over time for each of the dowel bar placements in the outside wheel path. There was also little difference between each dowel arrangement for any given survey date. The relative uniformity of the dynamic k-value over time for the inside and outside wheel paths is most likely due to the asphalt subgrade, which provided a stronger subgrade.

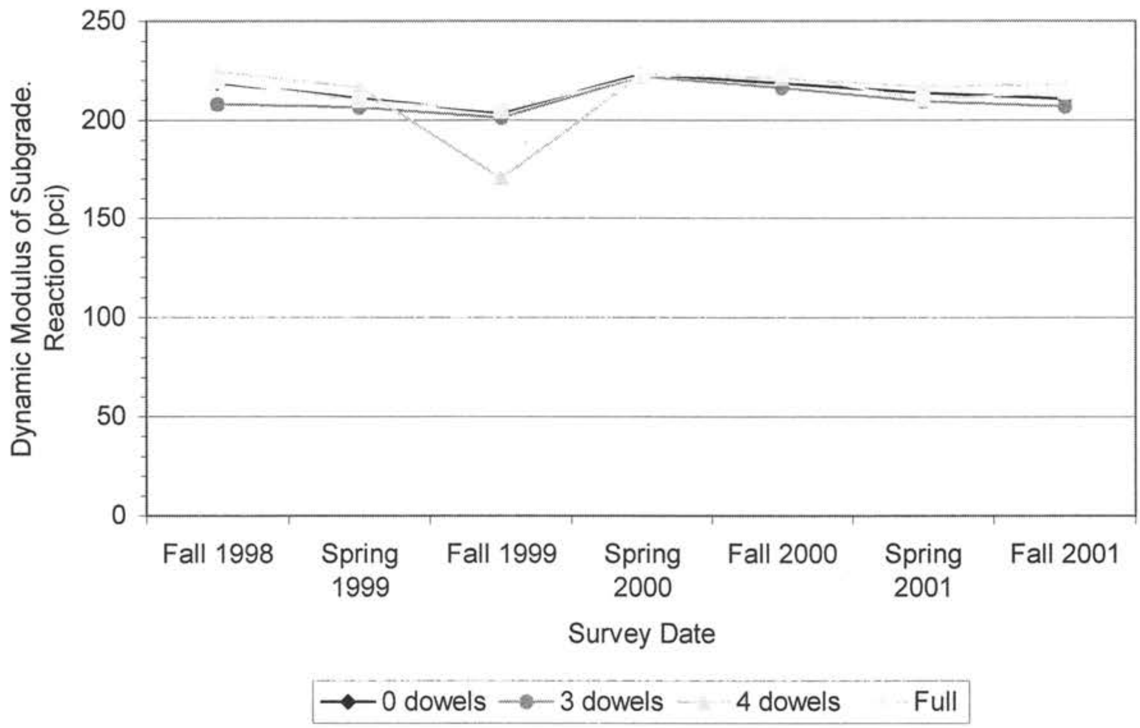


Figure 4.8 Average dynamic k-value vs. survey date for Rural site, inside wheel path.

The dynamic k-values over time in the Rural site for the inside wheel path was compared for each dowel arrangement in Figure 4.8. With the exception of four dowel bars in Fall 1999, there was relatively little difference between each dowel arrangement for any given survey date. There is a distinct difference in the dynamic k-value over the research period for each of the dowel bar placements. The fluctuation of the dynamic k-value over time is most likely caused by the weaker compacted soil subgrade, as compared to the Urban site.

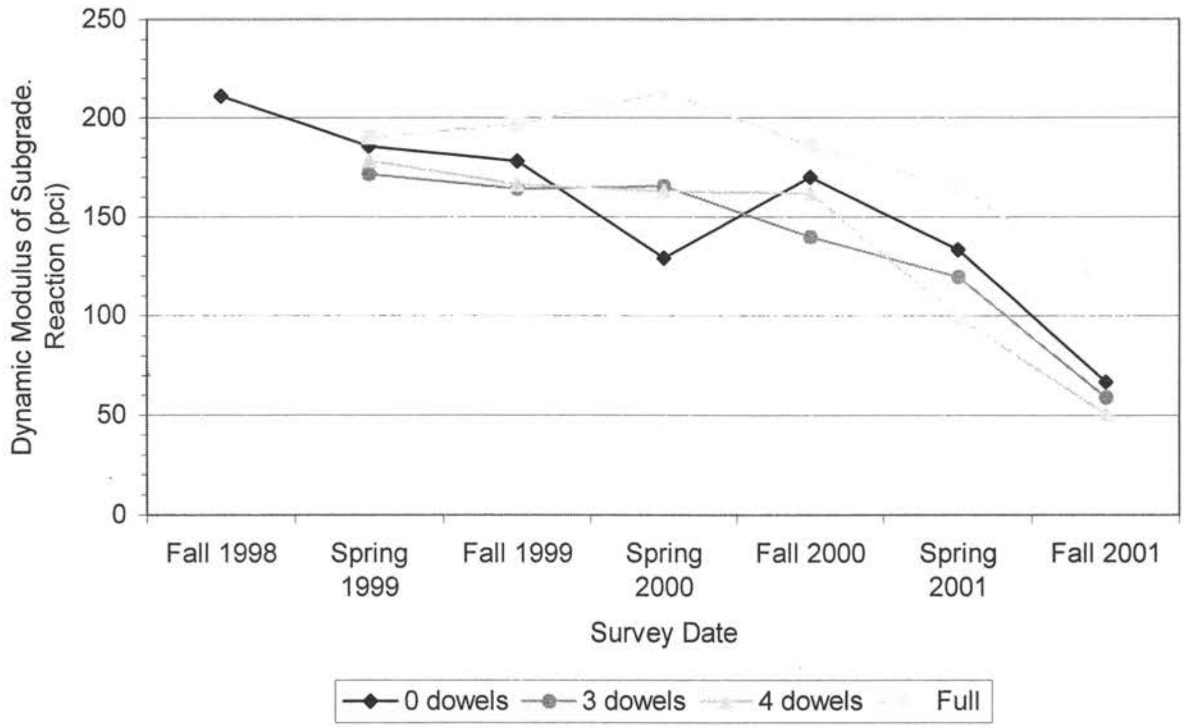


Figure 4.9 Average dynamic k-value vs. survey date for Rural site, outside wheel path.

The dynamic k-value over time in the Rural site for the outside wheel path was compared for each dowel arrangement in Figure 4.9. There was a distinct difference in the dynamic k-value over the research period for each of the dowel bar placements, with an overall increasing difference with increasing pavement age. There was also a decreasing trend over time for each dowel arrangement. As previously discussed, this decrease in dynamic k-value may be caused by the relatively unprotected shoulder, which is susceptible to water infiltration. Excess water infiltration would produce pore pressures in the soil, which would decrease the effective strength of the soil, thereby weakening the subgrade.

4.7.2 Joint deflections

4.7.2.1 Maximum joint deflection

Deflections measured at a joint are analyzed to determine the maximum joint deflection. Joint deflection is a function of the stiffness of the pavement system, including the subgrade, the concrete, and the efficiency of the load transfer devices. A stiffer subgrade and stronger concrete will resist larger joint deflections. High load transfer efficiency will distribute the load over a larger area, thereby decreasing the maximum joint deflection. The average maximum deflection for the Urban and Rural sites is tabulated in Table 4.8 and represented graphically in Figure 4.10 through Figure 4.14. Additional tabulated results and graphical representations of maximum deflection can be found in Appendix F.

Table 4.8 Average maximum deflection for research lifetime (in 1/1000 inch).

	Urban		Rural	
	Inside wheel path	Outside wheel path	Inside wheel path	Outside wheel path
Zero dowel bars	1.12	1.34	2.25	5.53
Three dowel bars	1.10	1.14	2.34	3.00
Four dowel bars	1.03	1.10	2.36	3.28
Full dowel bars	0.96	1.22	1.77	2.46

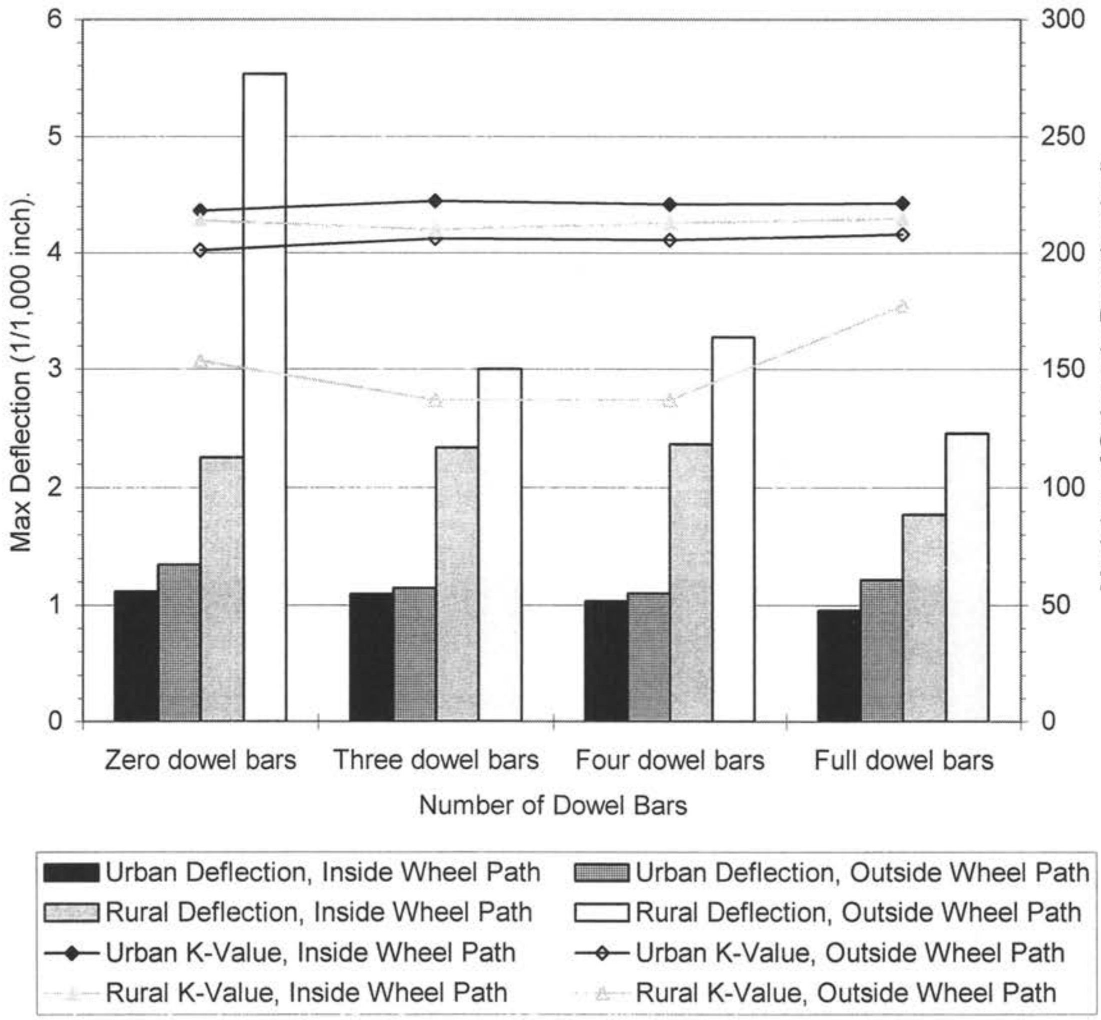


Figure 4.10 Average maximum deflection and average dynamic k-value vs. number of dowel bars.

Figure 4.10 compares the average maximum deflection and average dynamic modulus of subgrade reaction (dynamic k-value), over the research lifetime, for each dowel arrangement. For the inside and outside wheel paths in the Urban site there was not a distinct difference in the maximum deflection between dowel types. The inside wheel path in the Rural site also did not show a noticeable difference between zero, three, and four dowel bars, but did display slight decrease for the full dowel bar compliment. In contrast, the outside wheel path in the Rural site displayed a large difference between zero dowel bars and the other dowel placements. The full dowel bar compliment had the lowest maximum deflection for the outside wheel path in the Rural site. For all dowel placements, a large maximum deflection corresponded to a lower dynamic k-value. The relationship between the dynamic k-value and corresponding maximum deflection is magnified in the outside wheel path of the Rural site.

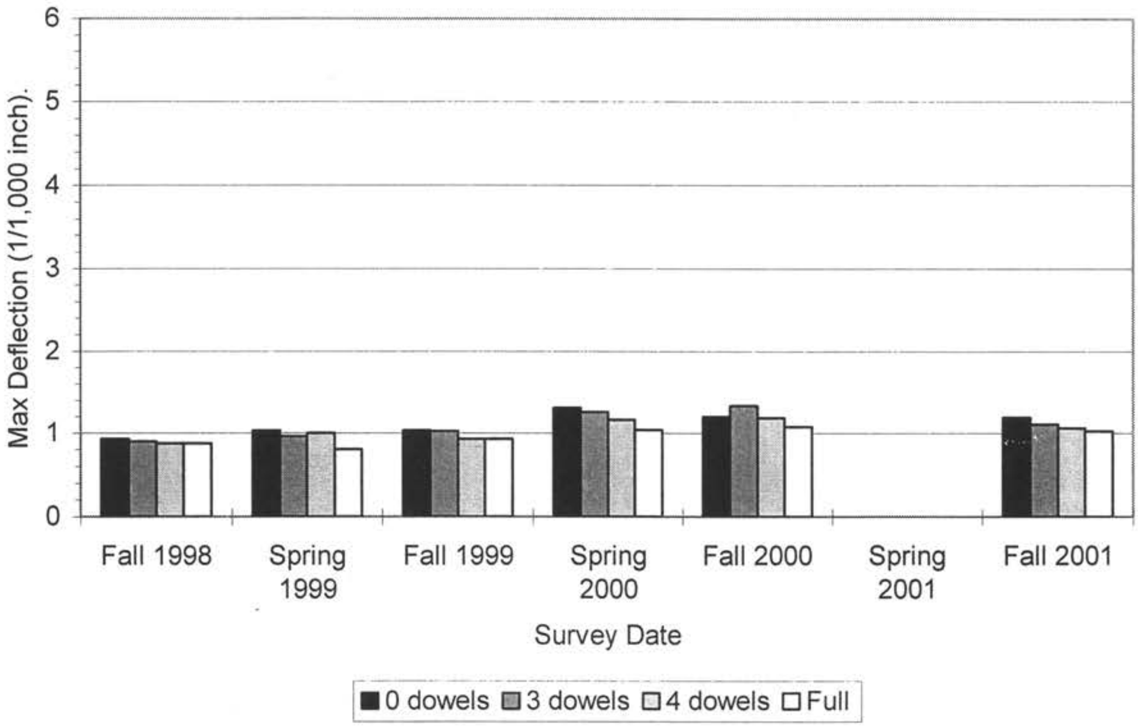


Figure 4.11 Average maximum deflection vs. survey date for the Urban site, inside wheel path.

Figure 4.11 compares the maximum deflection for each dowel arrangement over time in the Urban site for the inside wheel path. The maximum deflections over time for each dowel arrangement were relatively small. There appears to be little difference in the maximum deflection over time for each of the dowel bar placements. There was also little difference between each dowel arrangement for any given survey date.

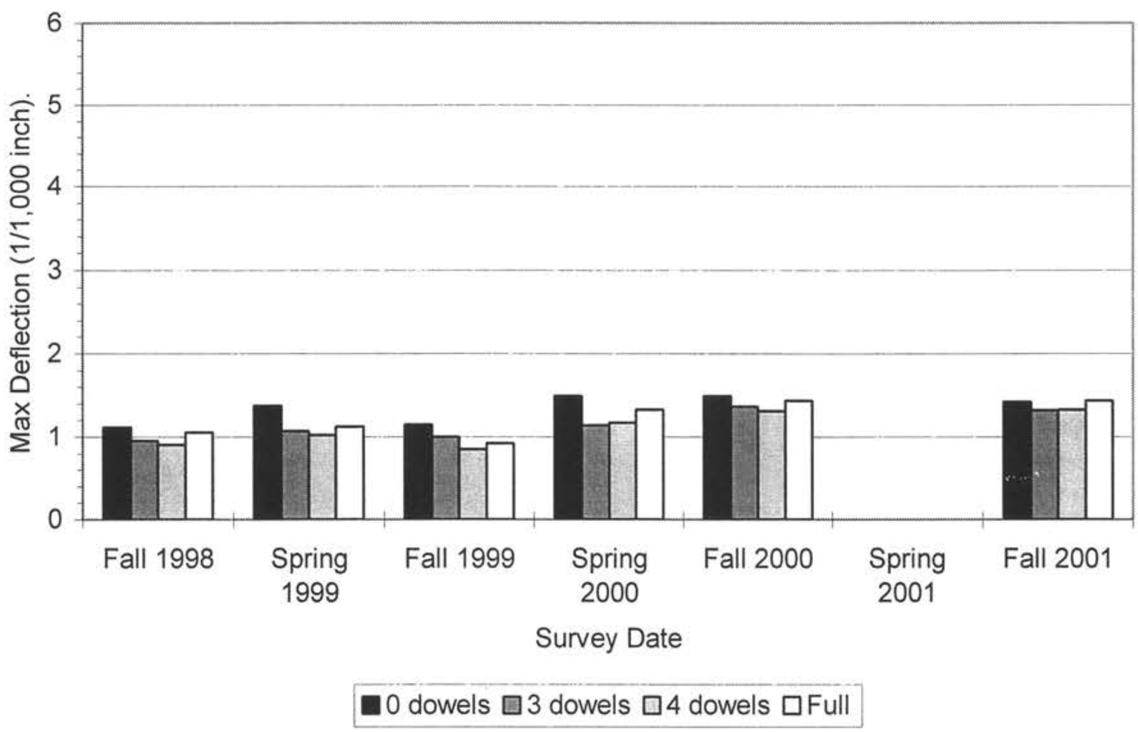


Figure 4.12 Average maximum deflection vs. survey date for the Urban site, outside wheel path.

Figure 4.12 compares the maximum deflection for each dowel arrangement over time in the Urban site for outside wheel path. As with the inside wheel path, there appears to be little difference in the maximum deflection over time for each of the dowel bar placements in the outside wheel path. There was also little difference between three, four, and full dowels for any given survey date, with the maximum deflection for the zero dowels slightly higher than the other dowel arrangements. In addition, the maximum deflections over time for each dowel arrangement were relatively small.

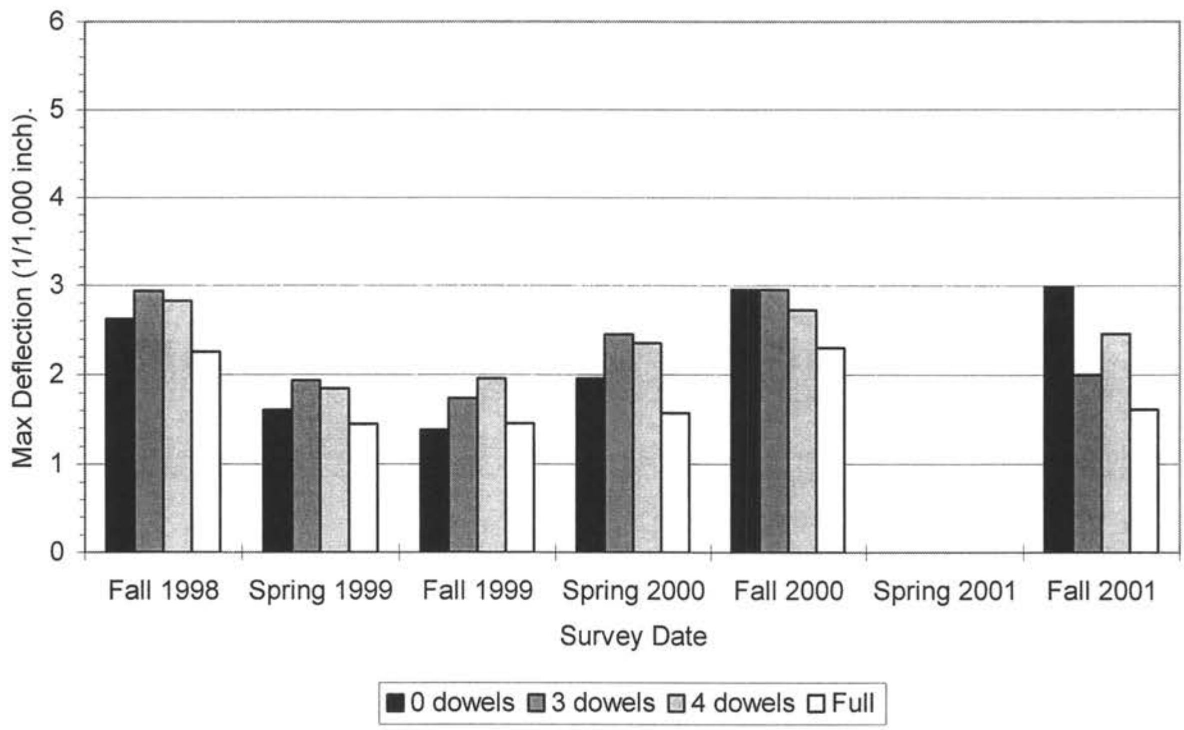


Figure 4.13 Average maximum deflection vs. survey date for the Rural site, inside wheel path.

The maximum deflection over time in the Rural site for the inside wheel path was compared for each dowel arrangement in Figure 4.8. Compared to the Urban site, the maximum deflections over time for each dowel arrangement in the Rural site were relatively large. There was a noticeable difference between the each dowel arrangement for any given survey date, with the full dowels exhibiting the smallest maximum deflection. There was also a distinct difference over time for each of the dowel bar placements. This fluctuation of the maximum deflection over time may be caused by variation in the stiffness of the subgrade over time.

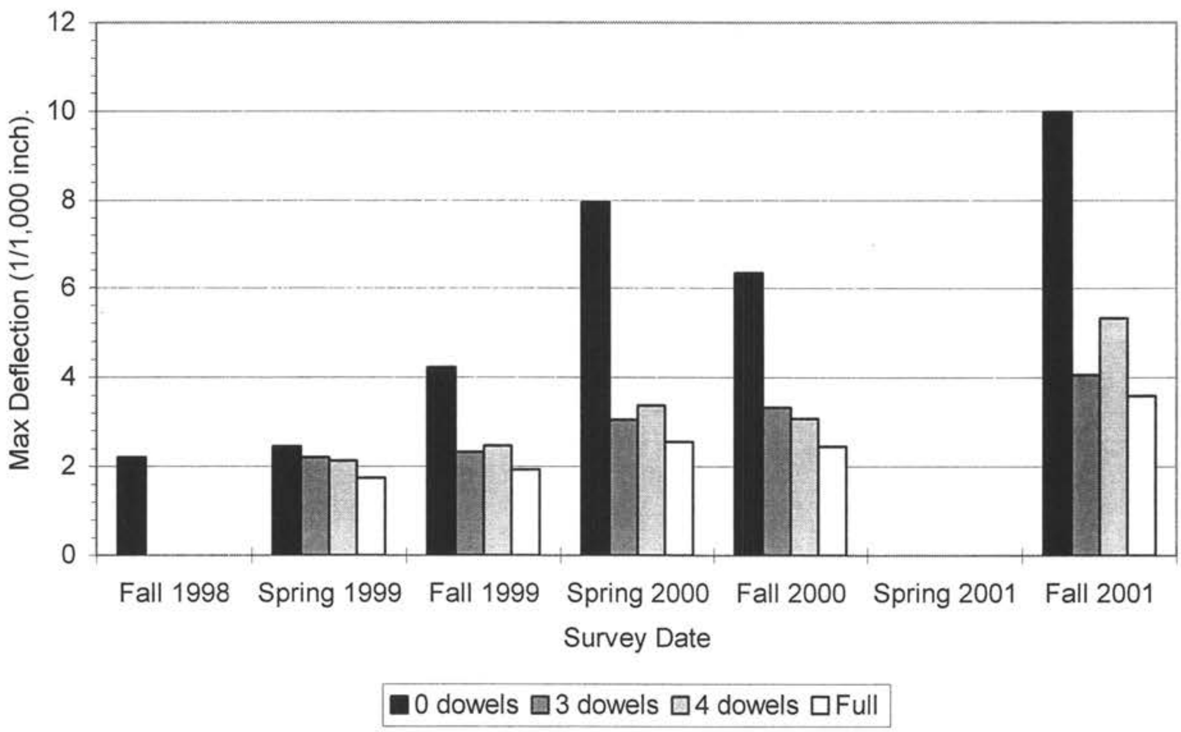


Figure 4.14 Average maximum deflection vs. survey date for the Rural site, outside wheel path.

The maximum deflection over time in the Rural site for the outside wheel path was compared for each dowel arrangement in Figure 4.14. There was a distinct difference in the maximum deflection between each dowel arrangement for any given survey date. The full dowels exhibited the smallest maximum deflections and the zero dowels exhibited the largest maximum deflections. All dowel arrangements exhibited increasing maximum deflection over time. It should be noted that the scale for the maximum deflection in Figure 4.14 is twice that of the maximum deflection scale in Figure 4.11 through Figure 4.13.

There was a noticeable difference in the maximum deflection over time for zero dowel bars, with an overall increasing maximum deflection with increasing pavement age. There was also a noticeable difference in the maximum deflection over time for three, four, and full dowels, again, with an overall increasing maximum deflection with increasing pavement age.

There was also a decreasing trend over time for each dowel arrangement. As previously discussed, this decrease in maximum deflection may be caused by the relatively unprotected shoulder, which is susceptible to water infiltration. Excess water infiltration would produce pore pressures in the soil, which would decrease the effective strength of the soil, thereby weakening the subgrade.

4.7.2.2 Load transfer efficiency

The deflection of the pavement at the joint can be analyzed to determine load transfer efficiency. Load transfer efficiency is defined as the ratio of the deflection of an unloaded slab to the deflection of the adjacent loaded slab, in percent (Equation 4.5). The first velocity sensor (beneath the load) is placed on one side of a transverse joint, with the remaining three sensors positioned on the adjacent, unloaded slab.

$$\text{Load transfer efficiency} = \frac{\text{Sensor 2}}{\text{Sensor 1}} * 100\% \qquad \textit{Equation 4.5}$$

where: Sensor 1 = Deflection at sensor located directly beneath the applied load
Sensor 2 = Deflection at sensors located at 12 inches from the applied load

ANOVA analyses were performed to determine the significance between the load transfer efficiency and season (spring or fall), location within the lane (inside or outside wheel path), lane direction (northbound or southbound for the Rural site, westbound or eastbound for the Urban Site), and dowel arrangement. Additional ANOVA analyses were conducted to compare the effect of the full compliment of dowel bars with the three alternative dowel arrangements.

The statistical results are tabulated in Table 4.9. The average load transfer efficiency for the Urban and Rural sites is tabulated in Table 4.10 and represented graphically in Figure 4.15 through Figure 4.19. Additional tabulated results and graphical representations of the load transfer efficiency can be found in Appendix G. Graphs comparing the relationship between the average load transfer and joint opening, joint faulting, modulus of subgrade reaction, and joint deflection for each dowel arrangement can also be found in Appendix G.

Table 4.9 Significance levels for average load transfer over research lifetime.

	Sample size	Average load transfer, (%)	Significance	Statistically significant?
Urban	1141	85.69		
Season			0.0654	no
Location within the lane			<0.0001	no
Lane direction			0.7464	yes
Dowel arrangement			<0.0001	yes
0 and full			<0.0001	yes
3 and full			<0.0001	yes
4 and full			<0.0001	yes
Rural	2053	75.25		
Season			0.0277	yes
Location within the lane			<0.0001	yes
Lane direction			<0.0001	yes
Dowel arrangement			<0.0001	yes
0 and full			<0.0001	yes
3 and full			<0.0001	yes
4 and full			<0.0001	yes

The test results for the Urban site indicate:

- The season does not have a significant effect on load transfer.
- The location within the lane (inside or outside wheel path) has a significant effect on load transfer.
- The direction of traffic does not have a significant effect on load transfer.
- Dowel arrangement has a significant effect on load transfer.
- There is a significant difference between the effect of the full dowel compliment and each of the other dowel placements on load transfer.

The test results for the Rural site indicate:

- Load transfer is significantly affected by season, location within the lane, lane direction, and dowel arrangement.
- There is a significant difference between the effect of the full dowel compliment and each of the other dowel placements on load transfer.

Table 4.10 Average load transfer over research lifetime.

	Urban		Rural	
	Inside wheel path	Outside wheel path	Inside wheel path	Outside wheel path
Zero dowel bars	83.54	86.21	70.08	58.30
Three dowel bars	79.65	88.51	74.33	81.58
Four dowel bars	81.14	90.04	69.84	80.79
Full dowel bars	86.25	90.16	82.62	84.48

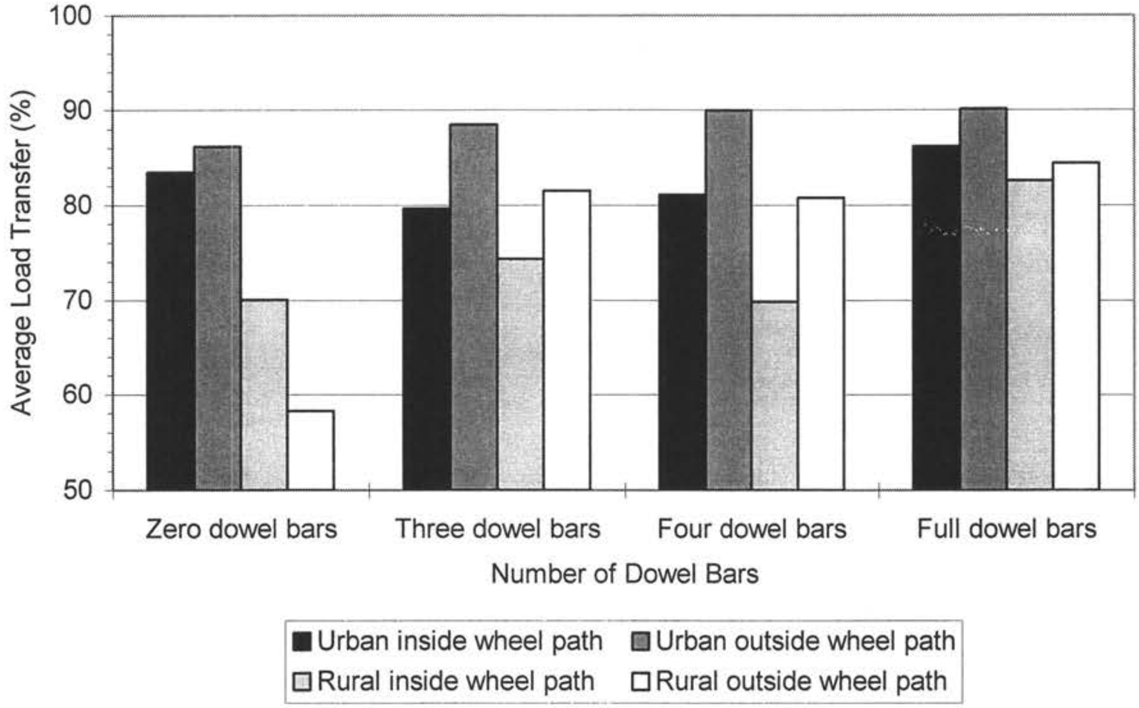


Figure 4.15 Average load transfer over research lifetime vs. number of dowel bars.

Figure 4.15 compares the average load transfer over the research lifetime for each dowel arrangement. The Urban site appears to exhibit higher load transfer than the Rural site for each dowel arrangement. With the exception of zero dowels in the Rural site, the outside wheel path exhibited higher load transfer for all dowel placements. The load in the inside wheel path can be transferred through the longitudinal tie bars to the adjacent lane, which would cause the load transfer in the inside wheel path to appear to lower than the outside wheel path.

It appears that the load transfer in the outside path, in both the Urban and Rural sites, is comparable for three, four, and full dowels. The load transfer in the inside wheel path, for the Rural site, is not comparable for any dowel bar arrangement. In contrast, the load transfer in the inside wheel path, for the Urban site, is comparable for all dowel bar arrangements.

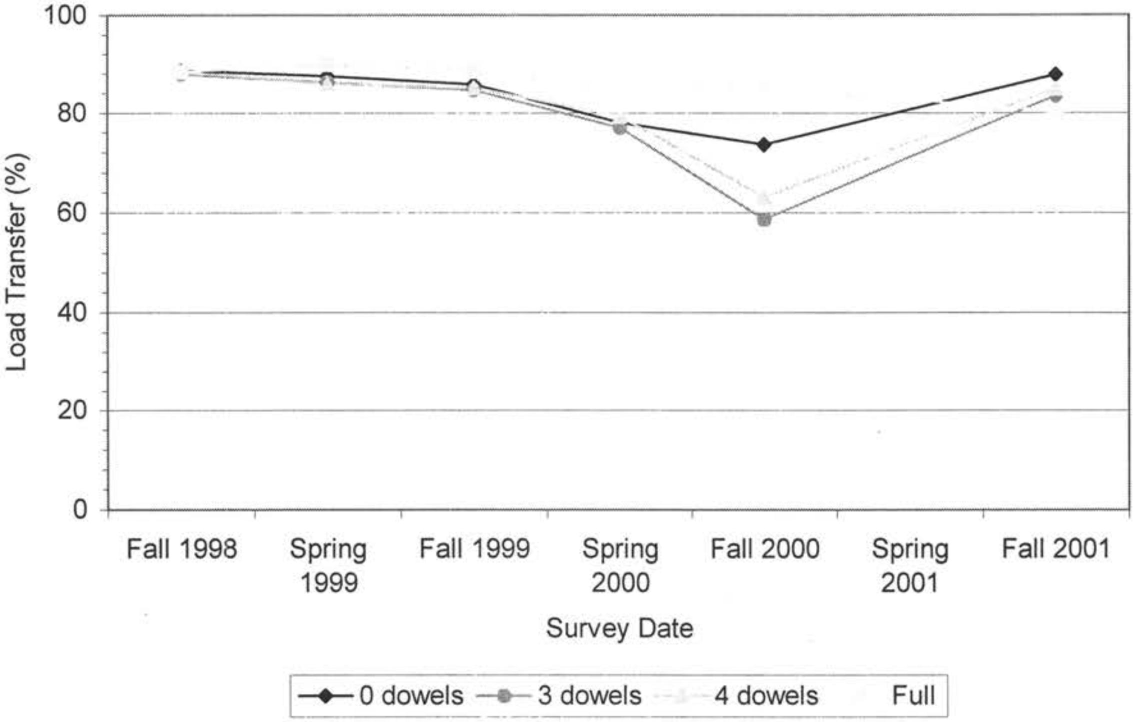


Figure 4.16 Average load transfer vs. survey date for the Urban site, inside wheel path.

Figure 4.16 compares the load transfer for each dowel arrangement over time in the Urban site for the inside wheel path. There appears to be little difference between each dowel arrangement for any given survey date, with the exception of zero dowel bars in Fall 2000. There was also little difference in the load transfer over time for each of the dowel bar arrangements, again with the exception of zero dowel bars in Fall 2000.

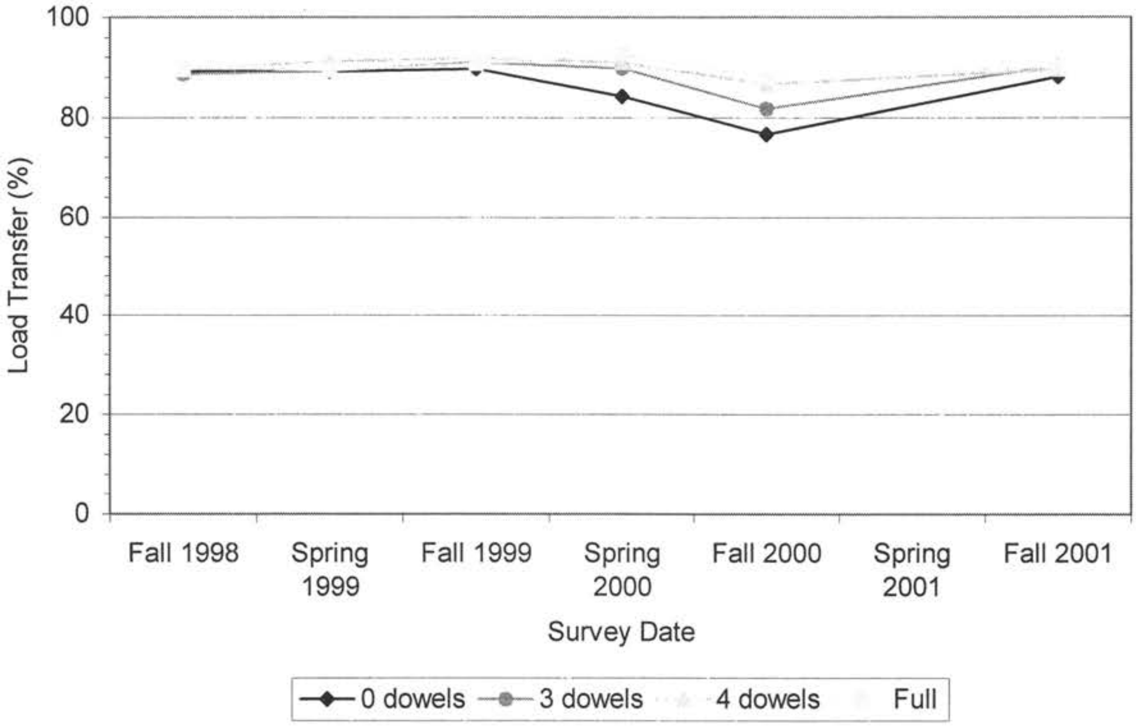


Figure 4.17 Average load transfer vs. survey date for the Urban site, outside wheel path.

Figure 4.17 compares the load transfer for each dowel arrangement over time in the Urban site for outside wheel path. Similar to the inside wheel path, there appears to be little difference in the load transfer over time for each of the dowel bar placements in the outside wheel path. There was also little difference between each dowel arrangement for any given survey date.

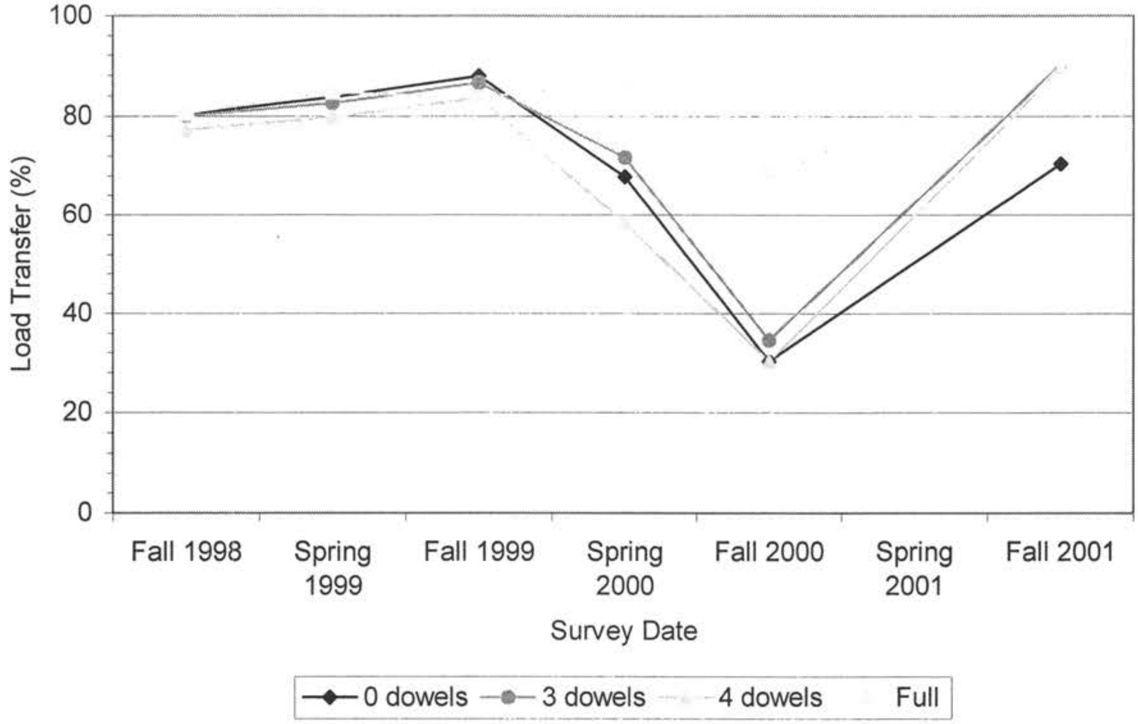


Figure 4.18 Average load transfer vs. survey date for the Rural site, inside wheel path.

The load transfer over time in the Rural site, for the inside wheel path was compared for each dowel arrangement in Figure 4.18. After Fall 1999, there was a noticeable difference between each dowel arrangement for any given survey date, with the exception of three four and full dowels in Fall 2001. There is a distinct difference in the load transfer over the research period for each of the dowel bar placements. The weaker subgrade in the Rural site appeared to exaggerate the effect of the different dowel placements.

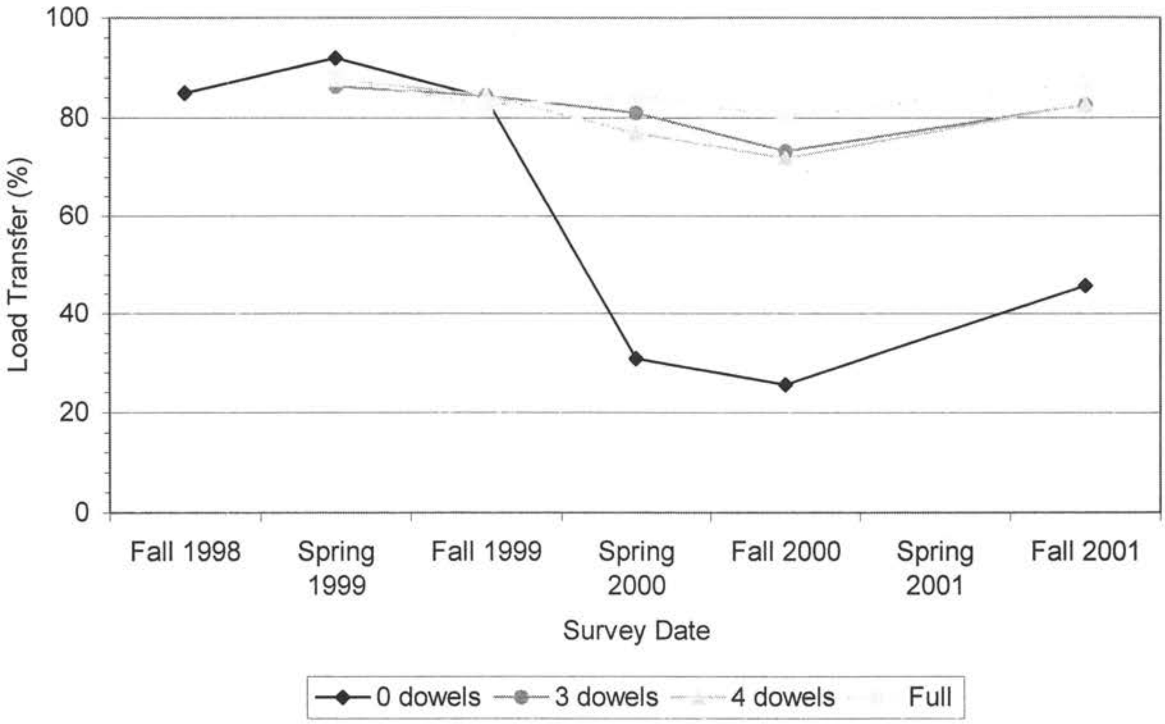


Figure 4.19 Average load transfer vs. survey date for the Rural site, outside wheel path.

The load transfer over time in the Rural site, for the outside wheel path was compared for each dowel arrangement in Figure 4.19. There was a noticeable difference in the load transfer over time for each dowel bar arrangements, with a distinct difference observed for the zero dowel section. As previously discussed, this decrease in load transfer may be caused by the weaker subgrade in the Rural site.

4.8 Material Savings

The objective of this research is to evaluate the potential for reducing the number of dowel bars across a transverse joint. Reduction in the number of dowel bars in a concrete pavement would have the potential to decrease the cost of construction. In addition, the use of fewer dowels would reduce potential problems associated with misaligned dowels. Table

4.11 demonstrates the reduction in the number of dowels per lane-mile. For 15-foot, 20-foot, and 25-foot joint spacings.

Table 4.11 Number of dowels per lane-mile.

Number of dowels per joint	Number of dowels per lane-mile		
	15 ft joint spacing	20 ft joint spacing	25 ft joint spacing
12	4,224	3,168	2,534
4	1,408	1,056	845
3	1,056	792	634

5. CONCLUSIONS

This research project involved the evaluation of alternative concrete pavement designs, specifically, altering the number and location of dowel bars in across a transverse joint. Previous laboratory research suggested that pavement performance might not be affected by reducing the number of dowel bars across a transverse joint. Therefore, this research project evaluated four dowel arrangements: 1) zero dowels, 2) three dowels in the outside wheel path, 3) four dowels in the outside wheel path, and 4) full basket of dowels (twelve). Two test sites were prepared each with the four dowel arrangements, one with a compacted soil subgrade (Rural site) the other with an asphalt concrete subgrade (Urban site).

The research objective was to evaluate the impact of the number of dowel bars and dowel location on pavement and joint performance. To satisfy this objective, an evaluation of the test sections was performed biannually (early fall or late summer and early spring) over a five-year testing period. Testing in the spring would allow the evaluation of pavement with a typically wet, weak foundation, whereas in the early fall the subgrade is typically dry and strong. In addition to the biannual pavement evaluation, a soil investigation was performed using in-situ soil classification from soil borings and consultation of US Department of Agriculture soil survey.

Biannual evaluation of both the Urban and Rural sites consisted of: 1) visual distress surveys, 2) joint opening measurements, 3) joint faulting measurements, and 4) deflection measurements using an Iowa DOT (Department of Transportation) Road Rater. The Road Rater deflection data, specifically deflection basin area, modulus of subgrade reaction, maximum deflection, and load transfer, was statistically and graphically analyzed.

5.1 Soil Survey

The Urban site was constructed on a Winterset silty clay loam with low permeability. The Winterset series consists of poorly drained soils formed in loess under natural vegetation. The Rural site, in contrast was constructed on three different, low permeability soils: Nira-Sharpsburg silty clay loam, Sharpsburg silty clay loam, and Clearfield silty clay loam. The Nira-Sharpsburg soil (60% Nira series, 35% Sharpsburg series) consists of moderately well drained soils formed in loess under native vegetation. The Sharpsburg series also consists of moderately well drained soils formed in loess under native vegetation. The Clearfield series, like the Winterset series, consist of poorly drained soils formed in three to six feet of loess over clayey glacial till, under native vegetation.

5.2 Joint Opening and Joint Faulting

Joint opening was analyzed to verify that the dowels had not become locked or frozen, i.e. the slabs were allowed to expand and contract due to temperature and moisture changes. The joint opening for each test section, in both the Urban and Rural sites, varied over time, indicating that the dowels had not become locked or frozen . This corresponds to the minimal pavement distress observed during the visual distress surveys.

Joint faulting was also analyzed to determine the effect of dowel bar arrangement with respect to site (Urban or Rural) and location within a lane (inside or outside wheel path). ANOVA analyses indicated that dowel arrangement had a significant effect on joint faulting for both sites. The largest faulting occurred, for Rural site, in the section without dowels, and for the Urban site, in the full dowel section. The faulting in the three and four dowel arrangements appeared to be similar.

5.3 Deflection Basin

The average deflection basin area is an indicator of structural soundness of a pavement over time, with a stable pavement tending to have an average area of 30-32 inches. The statistical analysis indicated that the deflection basin area is not significantly affected by dowel arrangement. In addition it was observed that during the research lifetime, the average deflection basin area was 31.23 inches and 30.44 inches for the Urban and Rural sites, respectively. These average deflection areas indicated that the pavement was structurally sound to date.

5.4 Dynamic Modulus of Subgrade Reaction

The dynamic modulus of subgrade reaction is a measure of the stiffness of the subgrade and is a function of the subgrade material and moisture content. For a less stiff subgrade with high moisture conditions, pumping of the subgrade may occur. Pumping of subgrade would be manifested in a decrease in dynamic modulus of subgrade reaction. Freeze/thaw and saturated soil (without pumping) conditions could also cause a decrease in the dynamic modulus of subgrade reaction.

Graphical analyses did not indicate a trend of decreasing dynamic modulus of subgrade reaction over time in the Urban site. The Rural site, in contrast, displayed a decreasing trend in dynamic modulus of subgrade reaction in the outside wheel path. The outside wheel path has higher potential for water infiltration as compared to the inside lane, which could lead to pumping of the compacted soil base in the Rural site. The average dynamic modulus of subgrade reaction for the inside and outside wheel paths in the Urban site and the inside wheel path in the Rural site were in excess of 200 pci for the duration of research period. Whereas the average research lifetime modulus of subgrade reaction for the inside wheel path in the Rural site ranged from 137 pci, for three and four dowels, to 178 pci for the full dowel arrangement.

5.5 Maximum Joint Deflection

Joint deflection is a function of the stiffness of the pavement system, which includes the subgrade, concrete, and efficiency of load transfer devices. With the exception of the outside wheel path in the Rural site, the arrangement of dowels does not appear be the controlling variable in joint deflection. In contrast, it appears that the dynamic modulus of subgrade reaction is a controlling variable in maximum deflection. For all dowel bar arrangements, the highest dynamic modulus of subgrade reaction corresponds to the lowest deflection.

In addition, the Rural site (with a weaker compacted soil subgrade) exhibited the larger joint deflections than the Urban site (with a stiffer asphalt subgrade). The outside wheel path in the Rural site displayed the largest joint deflections for all dowel arrangements, with zero dowel bar section experiencing the largest deflection (5.53 mils).

5.6 Load Transfer Efficiency

Load transfer efficiency is the ratio of the deflection of an unloaded slab to the deflection of the adjacent loaded slab. Load transfer is achieved by two mechanisms: aggregate interlock and load transfer devices. In the absence of load transfer devices load is transferred solely by aggregate interlock. It was observed that the average research lifetime load transfer for the zero dowel section in both the Urban and Rural sites was lower than the other three sections (three, four, and full). In the Rural site, the load transfer was significantly reduced in the outside wheel path (load transfer was approximately 58%).

In the Urban site the four dowel arrangements performed comparably, with the exception of zero dowels, in which the load transfer was only slightly lower. In contrast, only the three dowel and four dowel arrangements in the Rural site performed comparably. The full dowel section in the Rural site achieved the highest average load transfer during the research period, and the zero dowel section achieved the lowest average load transfer.

6. RECOMMENDATIONS

6.1 Design Recommendations

The results of this research project indicate that the stiffness of a subgrade magnifies the effect of dowel arrangement on a pavement. Therefore, for pavements with a weak subgrade(dynamic k-values less than 200), it is recommended to use the standard (full) dowel compliment. For pavements with strong/stiff subgrade (dynamic k-values greater than 220), three or four dowels in the outside wheel path will suffice. The results of this research indicate there is not a noticeable difference between three and four dowels. Additional investigation is needed to recommend a dowel bar arrangement, possibly with dowels in the inside wheel path, for moderately weak to moderately strong subgrades.

6.2 Further Study

It is recommended that the pavement evaluation for this research project continue to periodically monitor the pavement throughout its lifetime. This would enable the evaluation of the long-term effect of alternative dowel arrangements on the performance of a pavement with a strong or weak subgrade.

APPENDIX A: SOIL BORING LOGS

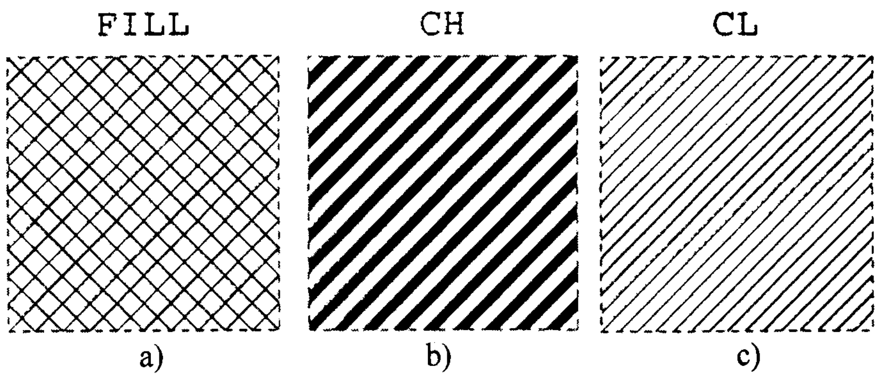


Figure A.1 Material descriptions.

A.1 Urban Site

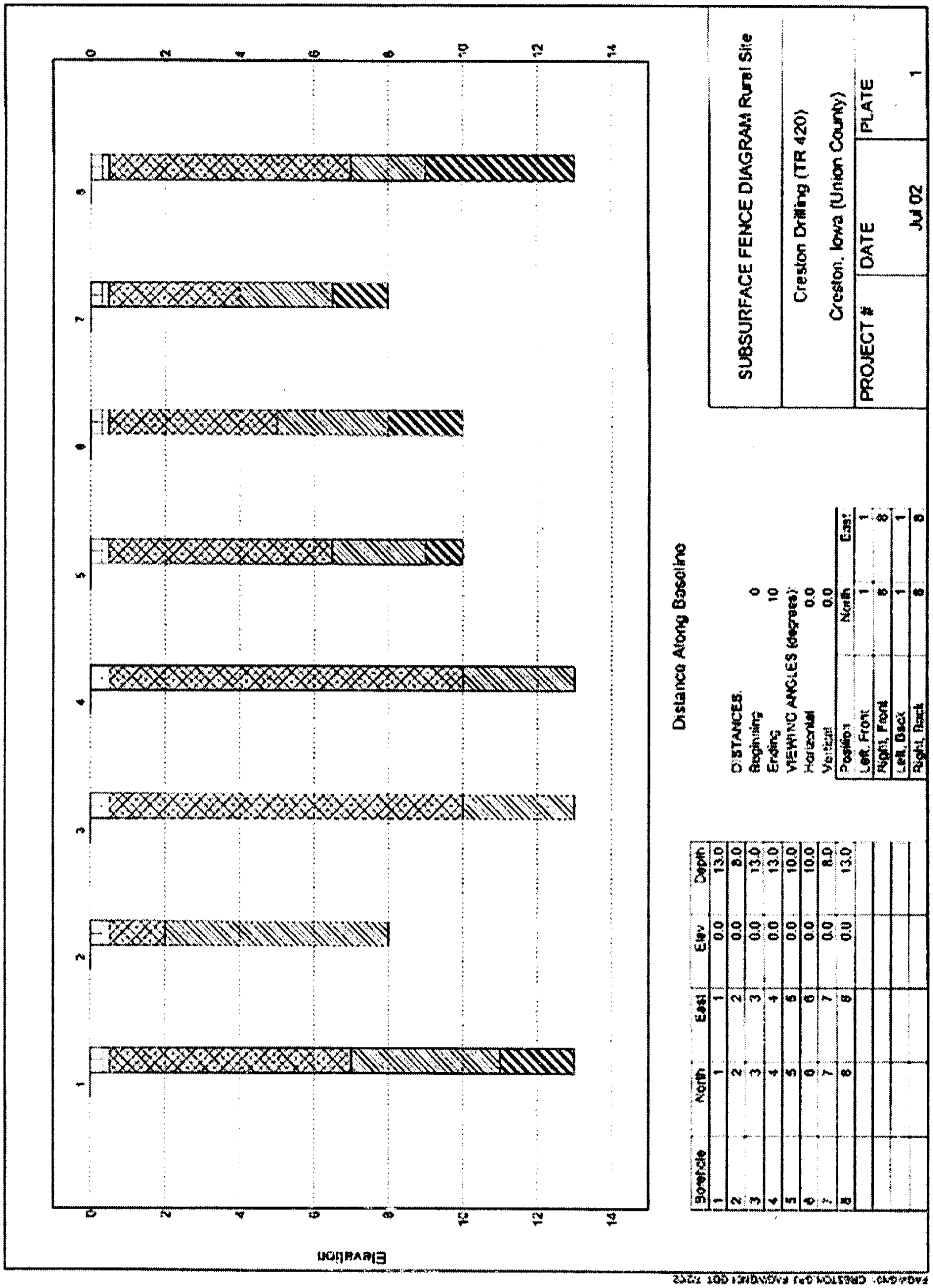


Figure A.2 Profile of soil borings, Urban Site.

Boring Number 1 **Lab Number** STA 178+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	13.0 feet	Client	
Drilling Method	Hollow Stem Auger (HSA)		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL) Fill
5									
									Grey silty clay with iron stains (CL) Loess
10									
									Grey clay (CH) Weathered glacial till
									Bottom of boring at 13 ft.
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/11/02

Figure A.3 Soil boring, STA: 178+50 RT 17.

Boring Number 2 **Lab Number** STA 182+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL) Fill
									Grey brown silty clay (CL)
5									Loess
									Bottom of boring at 8 ft.
10									
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/11/02

Figure A.4 Soil boring, STA: 182+50 RT 17.

Boring Number 3 **Lab Number** STA 186+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	13.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (∇), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL)
5									Fill
10									Black silty clay (CL)
									Loess
									Bottom of boring at 13 ft.
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/11/02

Figure A.5 Soil boring, STA: 186+50 RT 17.

Boring Number 4 **Lab Number** STA 190+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	13.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (∇), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL)
5									Fill
10									Grey silty clay (CL)
									Loess
									Bottom of boring at 13 ft.
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/1/02

Figure A.6 Soil boring, STA: 190+50 RT 17.

Boring Number 5 **Lab Number** STA 191+00 LT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	10.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL) Fill
5									
									Grey brown silty clay (CL) Loess
									Grey clay (CH) Weathered glacial till
10									Bottom of boring at 10 ft.
15									

TEST BORING CRESTON.GPJ IOWA ST.GDT 7/11/02

Figure A.7 Soil boring, STA: 191+00 LT 17.

Boring Number 6 **Lab Number** STA 188+50 LT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	10.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL) Fill
5									Grey silty clay with iron stains (CL) Loess
									Grey clay (CH) Weathered glacial till
10									Bottom of boring at 10 ft.
15									

Figure A.8 Soil boring, STA: 188+50 LT 17.

Boring Number 7 **Lab Number** STA 184+50 LT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ____ ft @ completion (▽). ____ ft @ ____ hrs. (▼). ____ ft @ ____ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - brown to black sandy clay (CL) Fill
5									Grey brown silty clay (CL) Loess
									Grey clay (CH) Weathered glacial till
10									Bottom of boring at 8 ft.
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 1/11/02

Figure A.9 Soil boring, STA: 184+50 LT 17.

Boring Number 8 **Lab Number** STA 180+50 LT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	13.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (∇). ______ ft @ ______ hrs. (▼). ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - black to grey silty clay (CL) Fill
5									
									Black silty clay (CL) Loess
									Grey brown silty clay (CL) Loess
10									
15									Bottom of boring at 13 ft.

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 1/11/02

Figure A.10 Soil boring, STA: 180+50 LT 17.

A.2 Rural Site

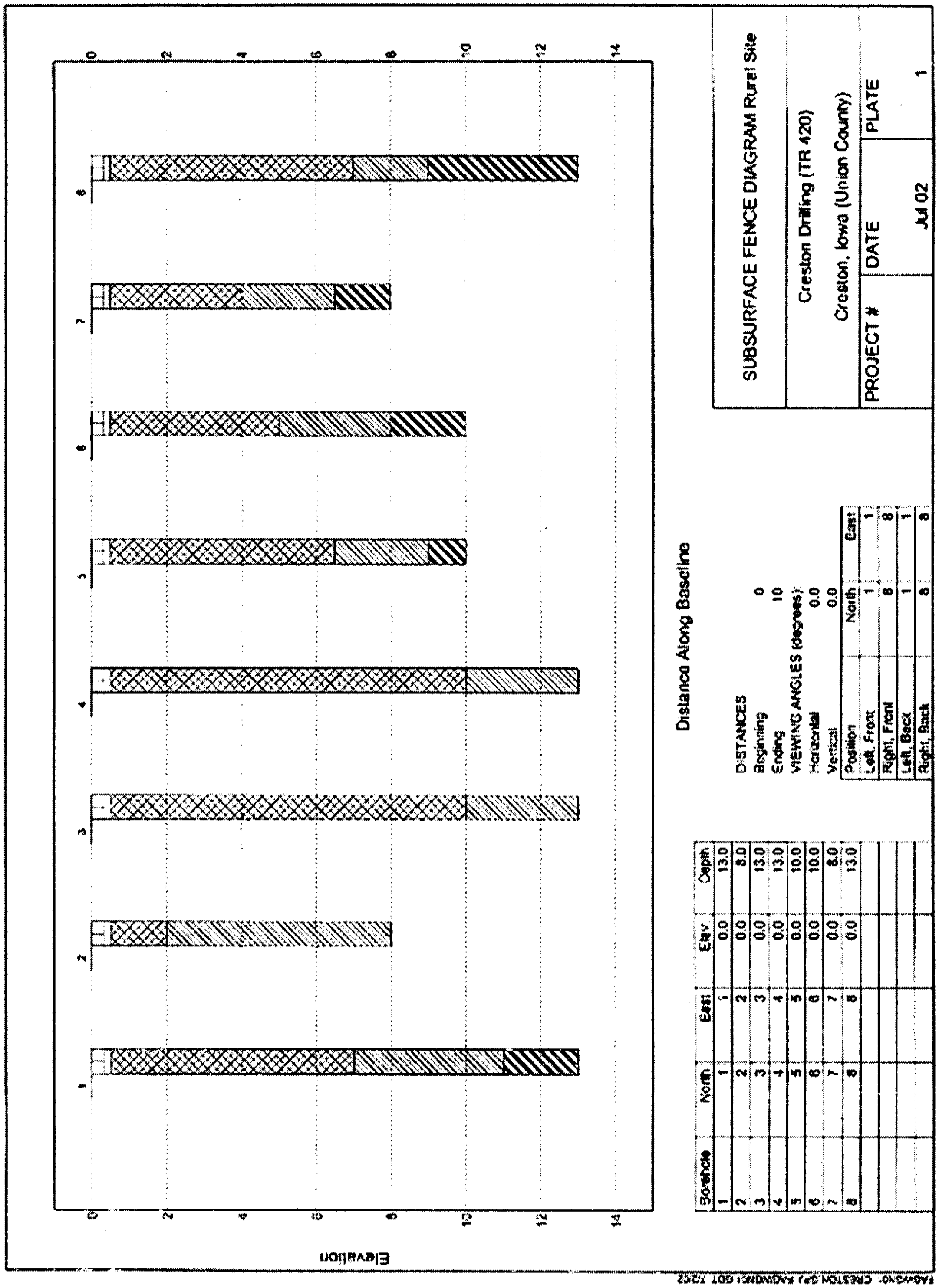


Figure A.11 Profile of soil borings, Rural Site.

Boring Number 9 **Lab Number** STA 78+50 LT 16

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
5									Grey brown silty clay (CL) Loess
10									Bottom of boring at 8 ft.
15									

TEST_BORING CRESTON.GPJ DWA_ST.GDT 7/11/02

Figure A.12 Soil boring, STA: 78+50 LT 16.

Boring Number 10 **Lab Number** STA 76+50 LT 16

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
									Black silty clay (CL) Loess
									Grey brown silty clay (CL)
5									Loess
									Bottom of boring at 8 ft.
10									
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/11/02

Figure A.13 Soil boring, STA: 76+50 LT 16.

Boring Number 11 **Lab Number** STA 74+50 LT 16

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	10.0 feet	Client	
Drilling Method	HSA		

Depth to Water ____ ft @ completion (▽), ____ ft @ ____ hrs. (▼), ____ ft @ ____ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown sandy silty clay (CL) Fill
									Black silty clay (CL) Loess
5									Grey brown silty clay (CL) Loess
10									Bottom of boring at 10 ft.
15									

TEST_BORING CRESTON.GPJ IOWA ST.GDT 7/1/02

Figure A.14 Soil boring, STA: 74+50 LT 16.

Boring Number 12 **Lab Number** STA 72+50 LT 16

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽). ______ ft @ ______ hrs. (▼). ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
									Grey brown silty clay (CL)
5									Loess
									Bottom of boring at 8 ft.
10									
15									

TEST_BORING CRESTON.GPJ IOWA_ST.GDT 7/11/02

Figure A.15 Soil boring, STA: 72+50 LT 16.

Boring Number 13 **Lab Number** STA 73+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	13.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (∇), ______ ft @ ______ hrs. (∇), ______ ft @ ______ hrs. (∇)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
5									Black silty clay (CL) Loess
									Grey brown silty clay (CL) Loess
10									
									Bottom of boring at 13 ft.
15									

TEST BORING CRESTON.GPJ IOWA ST.GDT 1/11/02

Figure A.16 Soil boring, STA: 73+50 RT 17.

Boring Number 14 **Lab Number** STA 75+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
									Grey brown silty clay (CL) Loess
5									
									Bottom of boring at 8 ft.
10									
15									

TEST BORING CRESTON.GPJ IOWA ST.GDT 7/1/02

Figure A.17 Soil boring, STA: 75+50 RT 17.

Boring Number 15 **Lab Number** STA 77+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
5									Grey brown silty clay (CL) Loess
									Bottom of boring at 8 ft.
10									
15									

TEST BORING CRESTON.GPJ IOWA ST.GDT 7/1/02

Figure A.18 Soil boring, STA: 77+50 RT 17.

Boring Number 16 **Lab Number** STA 79+50 RT 17

Date Drilled	12/20/01	Project	Creston Drilling (TR 420)
Surface Elevation	0		Creston, Iowa (Union County)
Depth Drilled	8.0 feet	Client	
Drilling Method	HSA		

Depth to Water ______ ft @ completion (∇), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Road Metal (crushed aggregate?)
									Fill - dark brown silty clay (CL) Fill
5									Grey brown silty clay (CL) Loess
									Bottom of boring at 8 ft.
10									
15									

Figure A.19 Soil boring, STA: 79+50 RT 17.

APPENDIX B: JOINT OPENING

Table B.1 Change in average joint opening for Rural site in southbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	43.07	10.41	37.05	29.11	20.45	21.06
3 dowels	61.35	28.72	61.59	52.69	42.19	19.86
4 dowels	46.71	9.74	46.08	36.99	27.93	-2.57
Full	48.61	5.54	36.40	20.06	11.79	135.15

Table B.2 Change in average joint opening for Rural site in northbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	-31.54	-58.13	-19.23	-50.22	-48.45	-48.05
3 dowels	9.59	-24.51	9.82	-9.47	-18.72	-9.86
4 dowels	13.11	-30.65	4.39	-11.24	-14.78	-0.02
Full	48.14	-4.95	38.83	23.19	12.41	14.48

Table B.3 Change in average joint opening for Urban site in westbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	1.20	-34.53	-18.96	-8.77	-34.58	-11.83
3 dowels	-4.10	-28.33	-30.23	-27.94	-41.90	-28.65
4 dowels	-20.37	-46.67	-34.55	-41.36	-53.17	-42.93
Full	-11.79	-34.98	-22.07	-39.00	-40.57	-13.80

Table B.4 Change in average joint opening for Urban site in eastbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	42.70	-10.73	20.45	1.16	2.34	9.04
3 dowels	37.27	36.16	26.11	10.36	20.02	25.39
4 dowels	59.82	17.62	38.25	26.75	38.17	28.33
Full	80.89	38.88	53.44	29.11	18.09	43.68

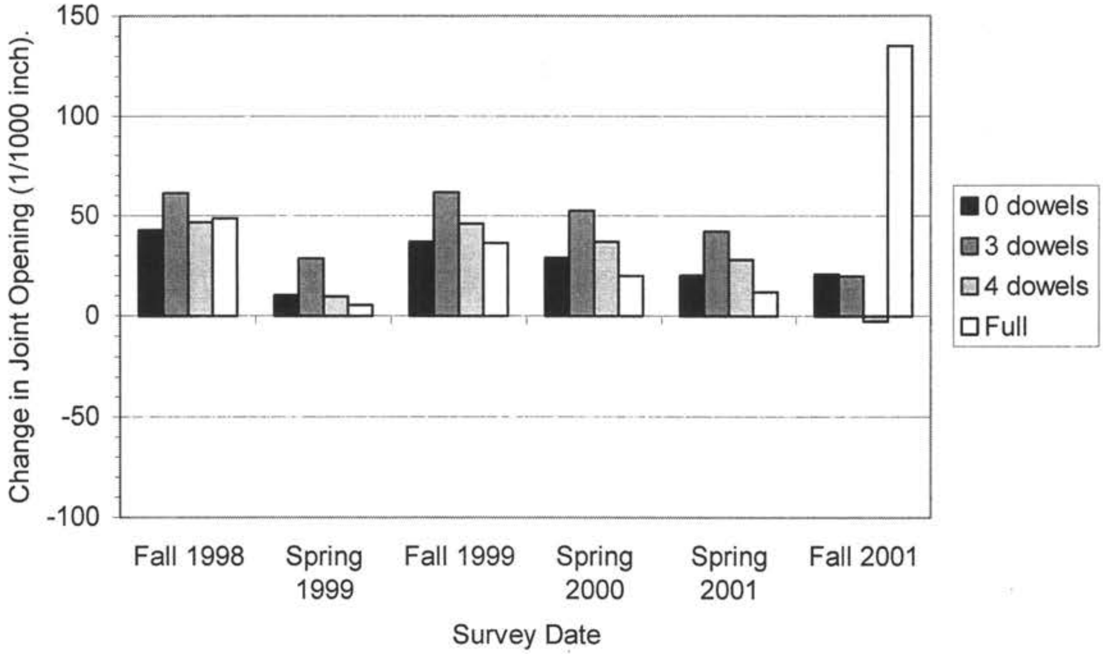


Figure B.1 Change in average joint opening for Rural site in southbound lane.

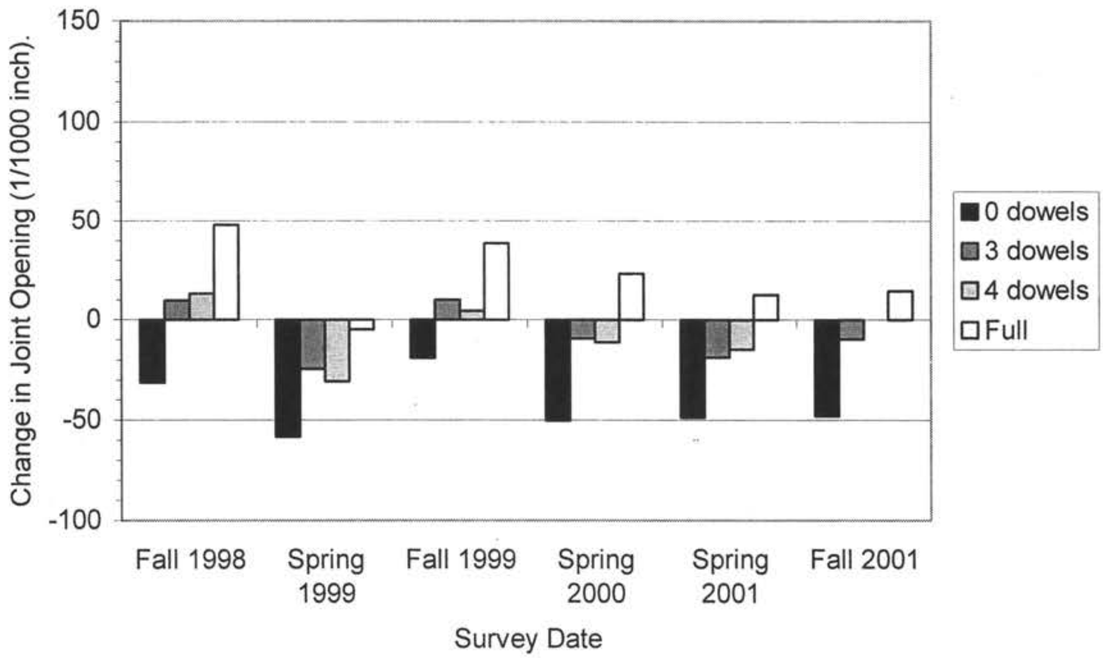


Figure B.2 Change in average joint opening for Rural site in northbound lane.

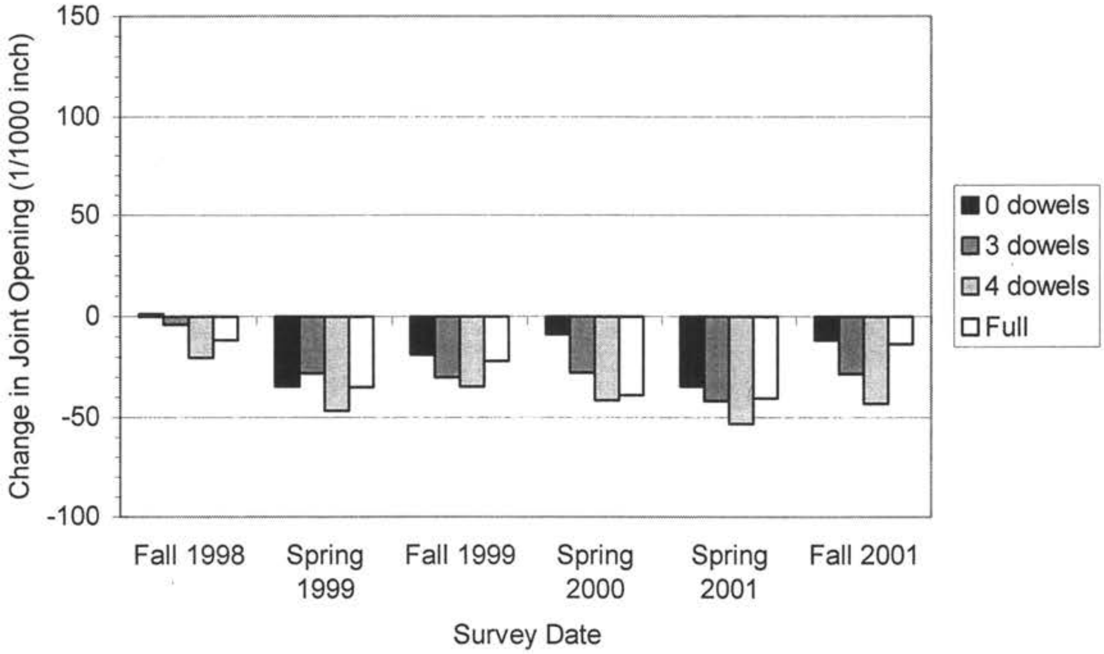


Figure B.3 Change in average joint opening for Urban site westbound lane.

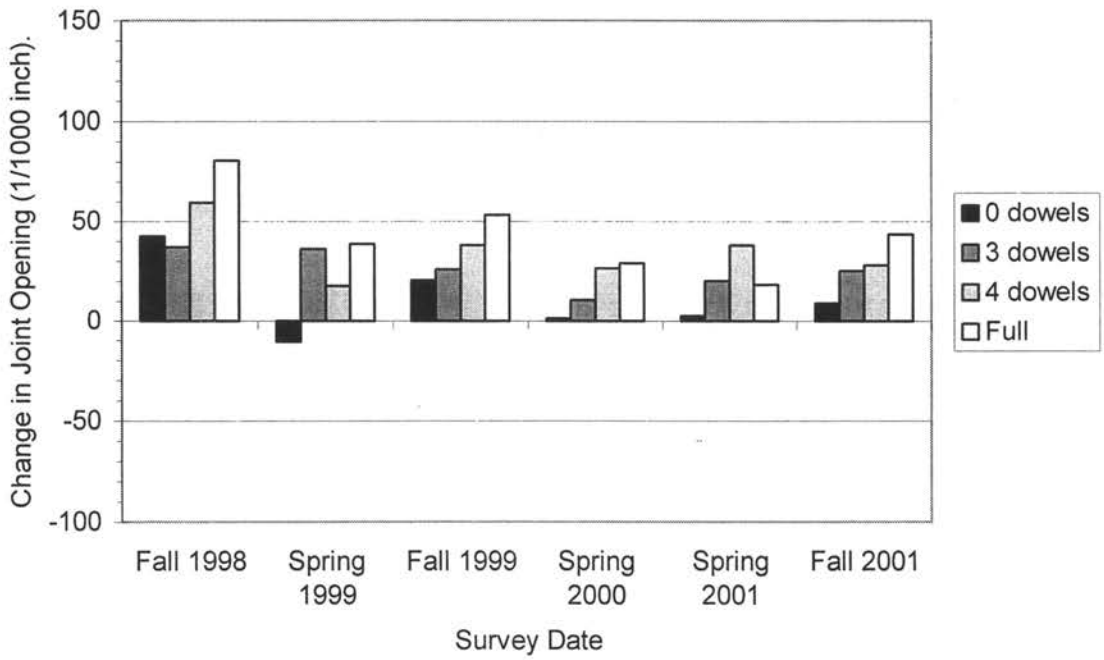


Figure B.4 Change in average joint opening for Urban site eastbound lane.

APPENDIX C: JOINT FAULTING

C.1 Inside Wheel Path

Table C.1 Average joint faulting for Rural site southbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	N.A.	-15.16	-22.24	-14.57	-20.87	-12.01
3 dowels	N.A.	-22.24	-19.49	-15.94	-27.56	-21.65
4 dowels	N.A.	-16.34	-7.68	-0.79	-20.67	-0.79
Full	N.A.	8.66	4.72	2.76	-0.39	-0.39

Table C.2 Average joint faulting for Rural site northbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	N.A.	-30.71	-21.65	-28.74	-24.02	-18.50
3 dowels	N.A.	-20.40	-9.31	-19.33	-18.25	-2.15
4 dowels	N.A.	7.48	-9.06	1.97	-1.97	1.57
Full	N.A.	6.69	1.57	0.79	15.35	-1.57

Table C.3 Average joint faulting for Urban site westbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	N.A.	-6.89	-17.13	-3.35	-9.25	1.18
3 dowels	N.A.	-0.39	0.00	2.17	-4.13	-0.39
4 dowels	N.A.	-6.30	0.98	-4.13	-11.22	2.36
Full	N.A.	-18.50	-18.31	-23.43	-32.68	-11.42

Table C.4 Average joint faulting for Urban site eastbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	N.A.	-32.68	-46.06	-41.73	-54.72	-40.16
3 dowels	N.A.	-1.07	-20.04	-1.79	-13.96	0.36
4 dowels	N.A.	-1.57	-12.99	0.79	-18.50	8.66
Full	N.A.	-30.71	-43.70	-42.91	-66.54	-41.73

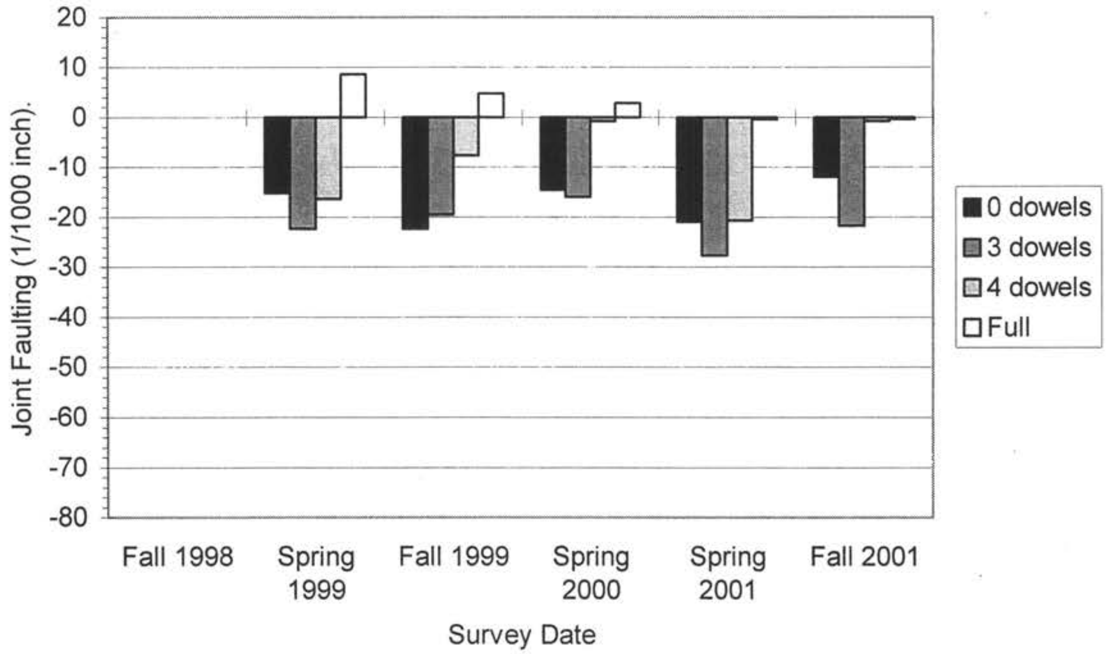


Figure C.1 Average joint faulting for Rural site southbound lane.

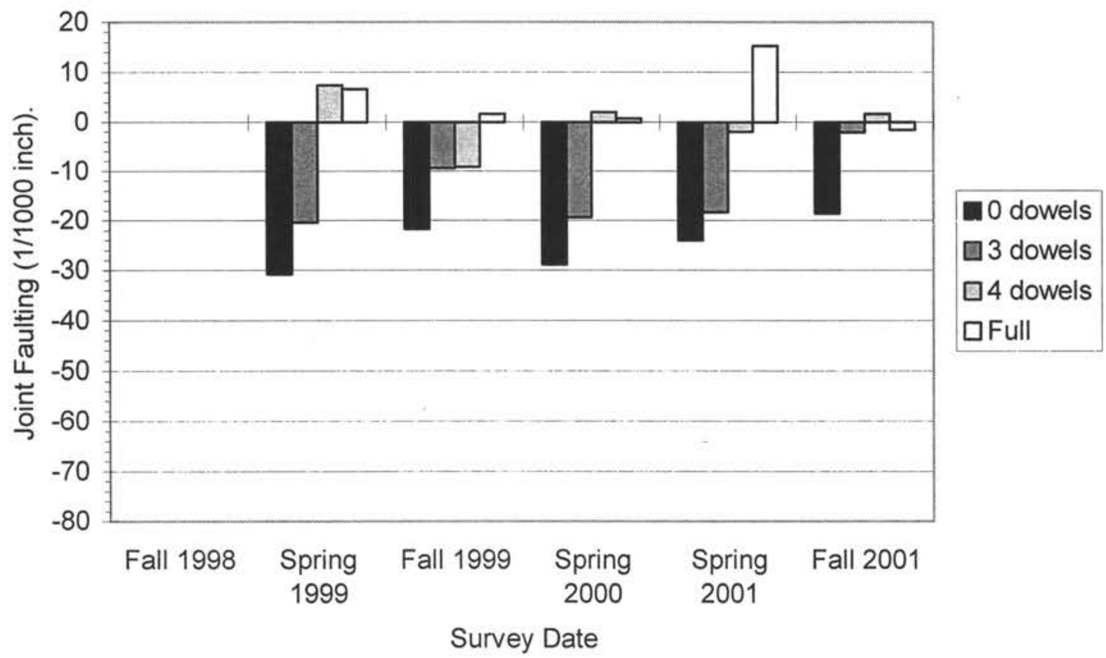


Figure C.2 Average joint faulting for Rural site northbound lane.

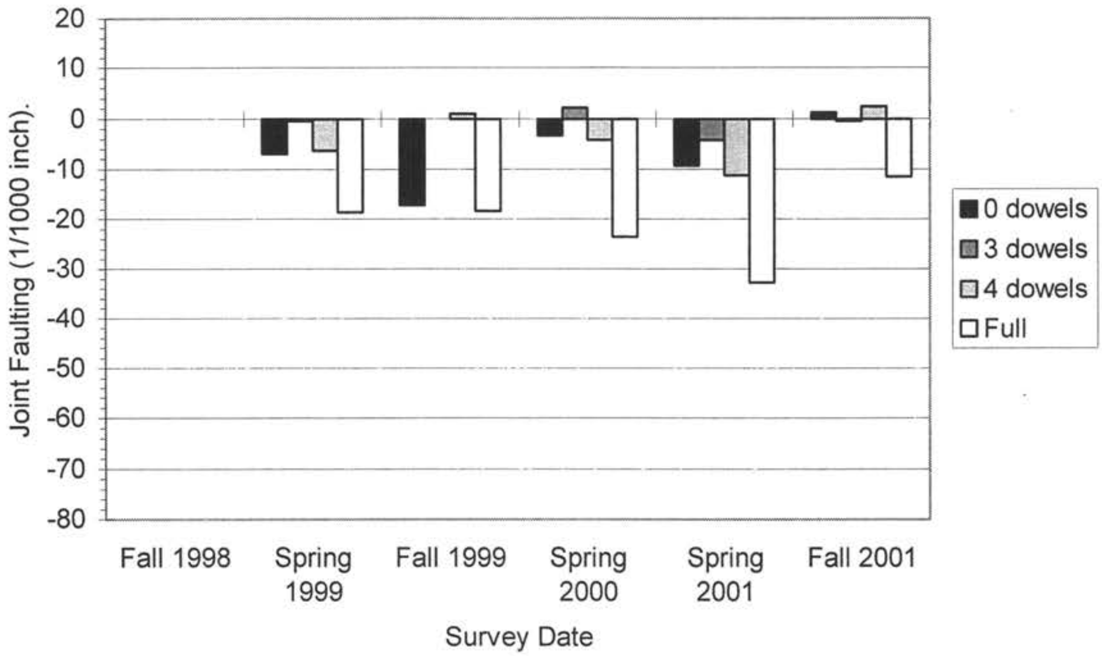


Figure C.3 Average joint faulting for Urban site westbound lane.

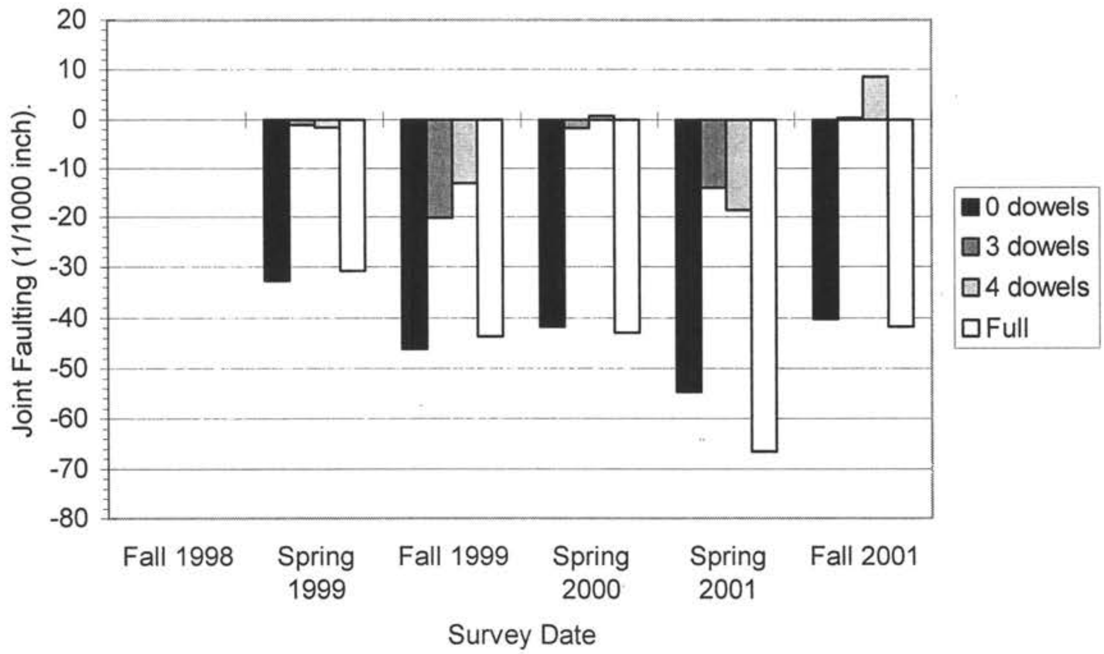


Figure C.4 Average joint faulting for Urban site eastbound lane.

C.2 Outside Wheel Path

Table C.5 Average joint faulting for Rural site southbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	1.38	-10.04	-37.99	-23.62	-10.83	-15.98
3 dowels	2.95	3.54	15.16	0.59	1.77	-11.22
4 dowels	9.06	0.20	8.46	1.18	-6.89	-6.89
Full	-2.56	-5.31	-9.65	-9.65	-20.28	-16.34

Table C.6 Average joint faulting for Rural site northbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	-6.30	-17.72	-14.17	-31.10	-17.32	-18.11
3 dowels	16.11	-13.96	-15.03	-20.76	-24.70	-17.18
4 dowels	-1.97	-24.80	-28.35	-22.05	-26.38	-24.80
Full	-8.27	-11.42	-3.15	-15.35	-21.26	-13.39

Table C.7 Average joint faulting for Urban site westbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	-8.86	-21.65	-36.42	-25.20	-10.83	-20.47
3 dowels	4.72	-25.98	-27.56	-22.05	-24.61	-14.57
4 dowels	1.57	-18.31	-17.32	-20.47	-23.23	-5.91
Full	-20.67	-31.10	-41.54	-35.04	-40.94	-36.42

Table C.8 Average joint faulting for Urban site eastbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Spring 2001	Fall 2001
0 dowels	-7.87	-21.26	-30.31	-26.38	-38.98	-26.38
3 dowels	-2.51	-11.45	-39.73	-12.17	-32.57	-12.88
4 dowels	0.79	-36.22	-37.01	-14.57	-36.22	-22.44
Full	-9.06	-16.93	-40.55	-22.83	-31.89	-23.23

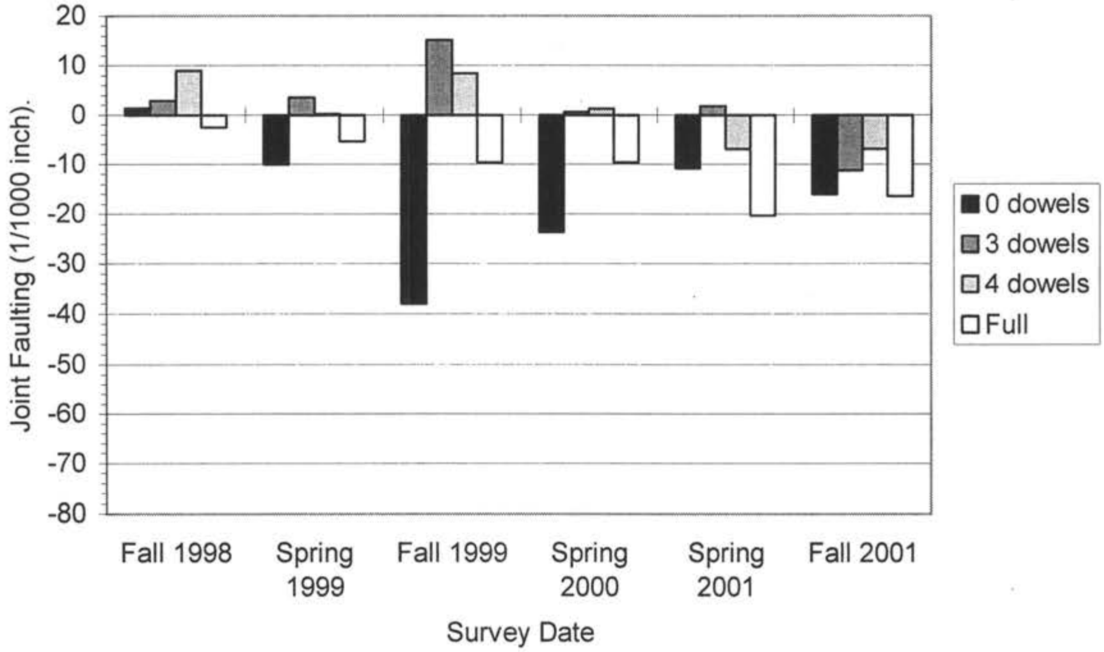


Figure C.5 Average joint faulting for Rural site southbound lane.

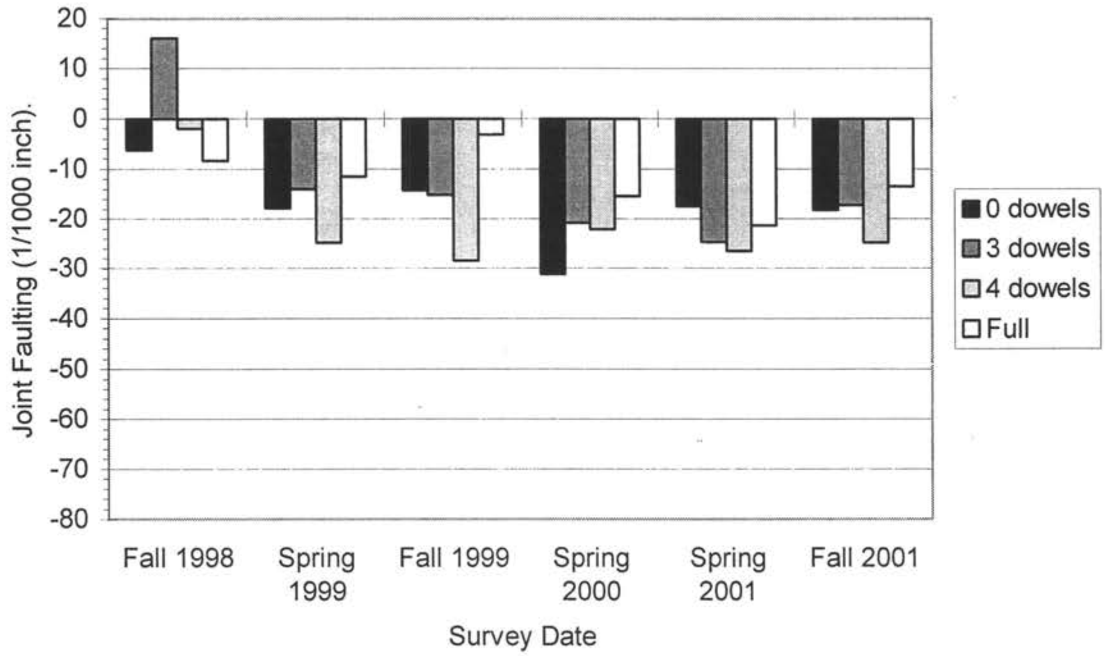


Figure C.6 Average joint faulting for Rural site northbound lane.

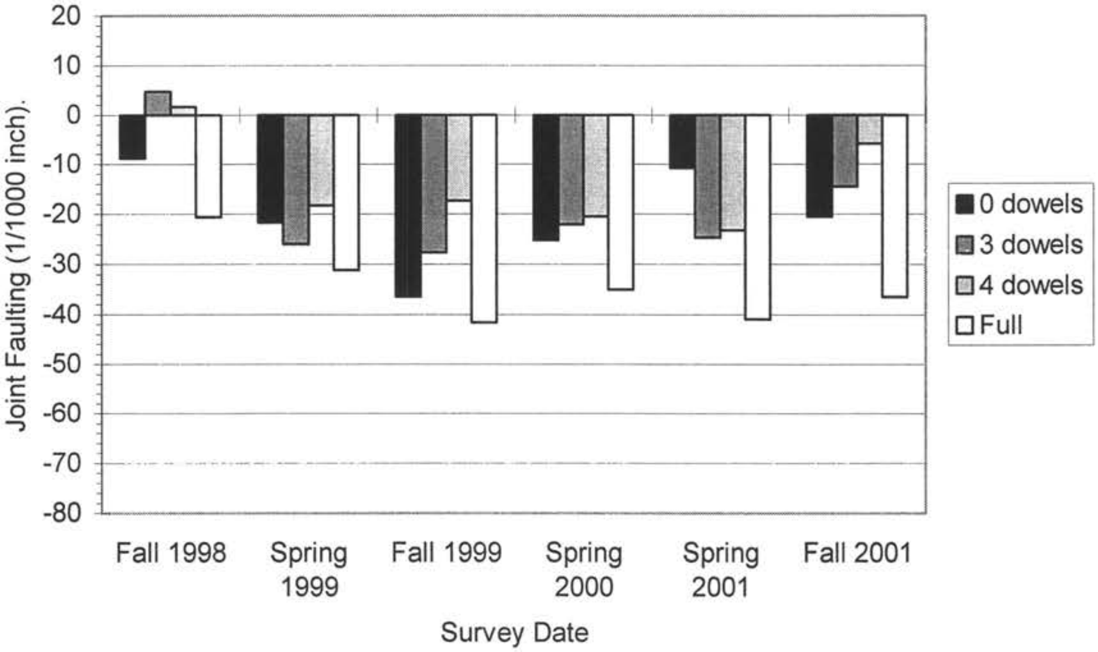


Figure C.7 Average joint faulting for Urban westbound lane.

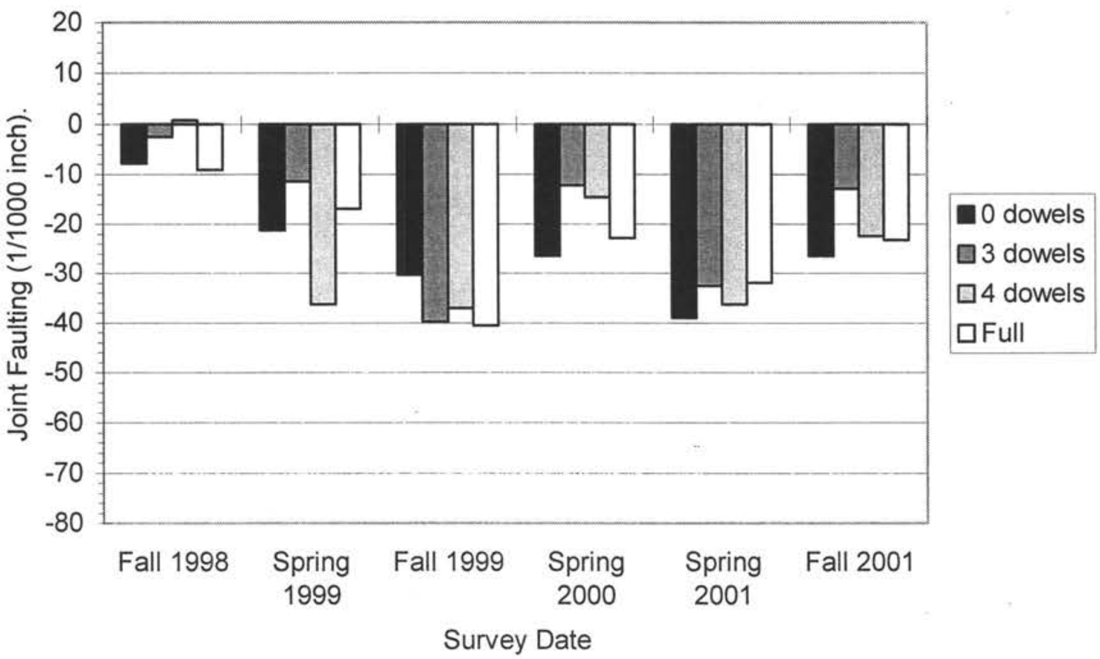


Figure C.8 Average joint faulting for Urban eastbound lane.

APPENDIX D: DEFLECTION BASIN AREA

Table D.1 Average deflection basin area for Rural site southbound lane (in inches).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	30.70	30.94	30.58	27.17	30.97	29.73	32.03
3 dowels	31.34	31.11	30.91	27.65	31.25	29.79	32.45
4 dowels	29.83	30.46	30.32	26.50	30.34	29.53	32.12
Full	30.69	30.74	30.81	27.55	30.50	29.31	31.69

Table D.2 Average deflection basin area for Rural site northbound lane (in inches).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	30.59	30.83	31.04	26.63	30.88	29.83	38.65
3 dowels	31.48	31.47	31.61	26.94	31.76	30.48	32.77
4 dowels	29.94	30.75	31.28	26.41	30.87	30.17	32.22
Full	30.30	30.55	30.66	27.55	30.60	29.54	31.69

Table D.3 Average deflection basin area for Urban site westbound lane (in inches).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	31.44	31.30	31.34	28.53	32.08	30.16	31.86
3 dowels	31.34	30.98	31.21	28.53	31.89	30.17	32.06
4 dowels	31.71	31.54	31.39	29.20	32.09	30.63	32.40
Full	31.16	31.10	30.74	28.93	31.53	29.77	31.97

Table D.4 Average deflection basin area for Urban site eastbound lane (in inches).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	31.78	31.84	32.11	30.21	31.15	30.52	32.72
3 dowels	31.38	31.96	31.70	29.31	32.07	30.19	32.50
4 dowels	31.78	32.09	31.84	30.14	32.21	30.55	32.59
Full	31.91	32.02	31.66	30.54	31.99	30.33	32.82

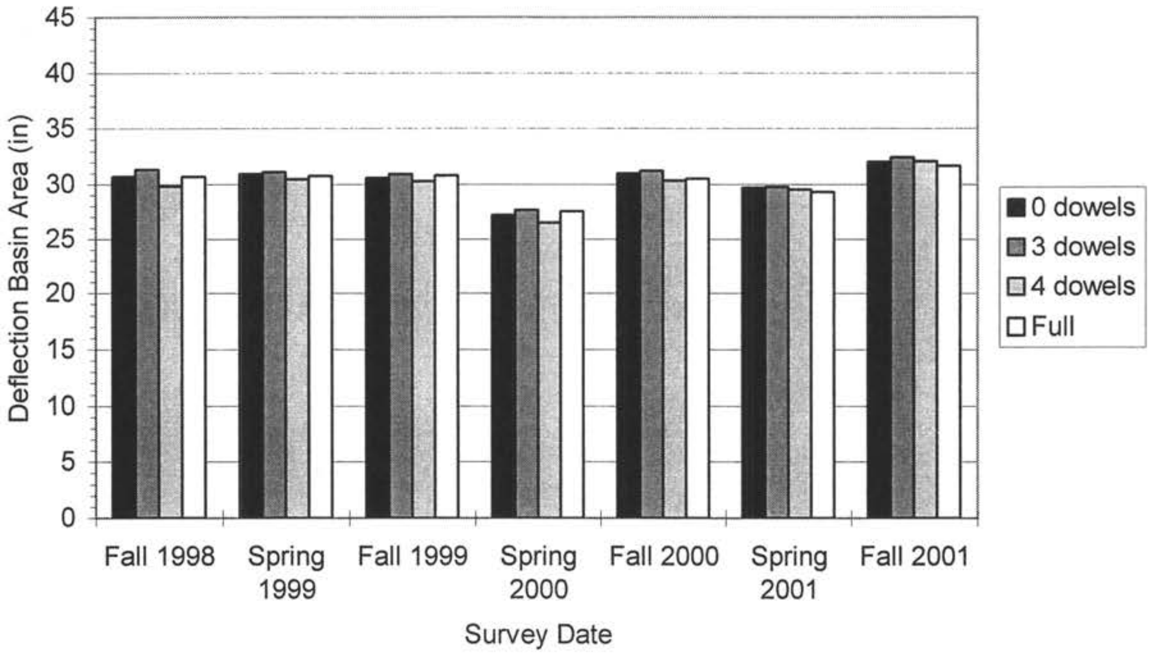


Figure D.1 Average deflection basin area for Rural site southbound lane.

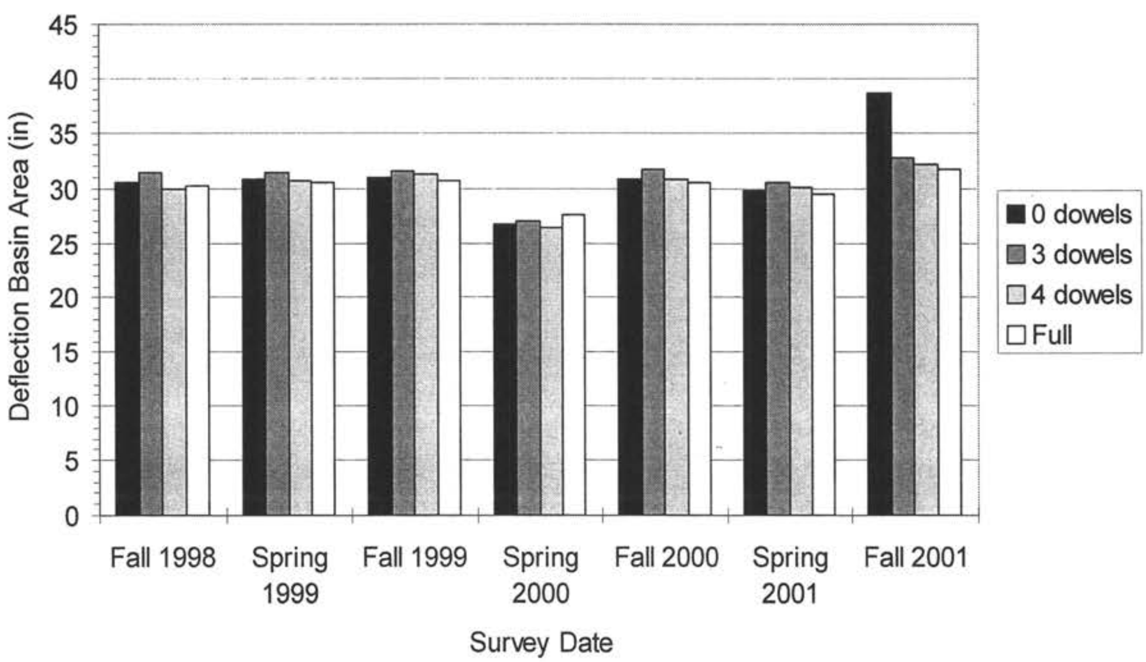


Figure D.2 Average deflection basin area for Rural site northbound lane.

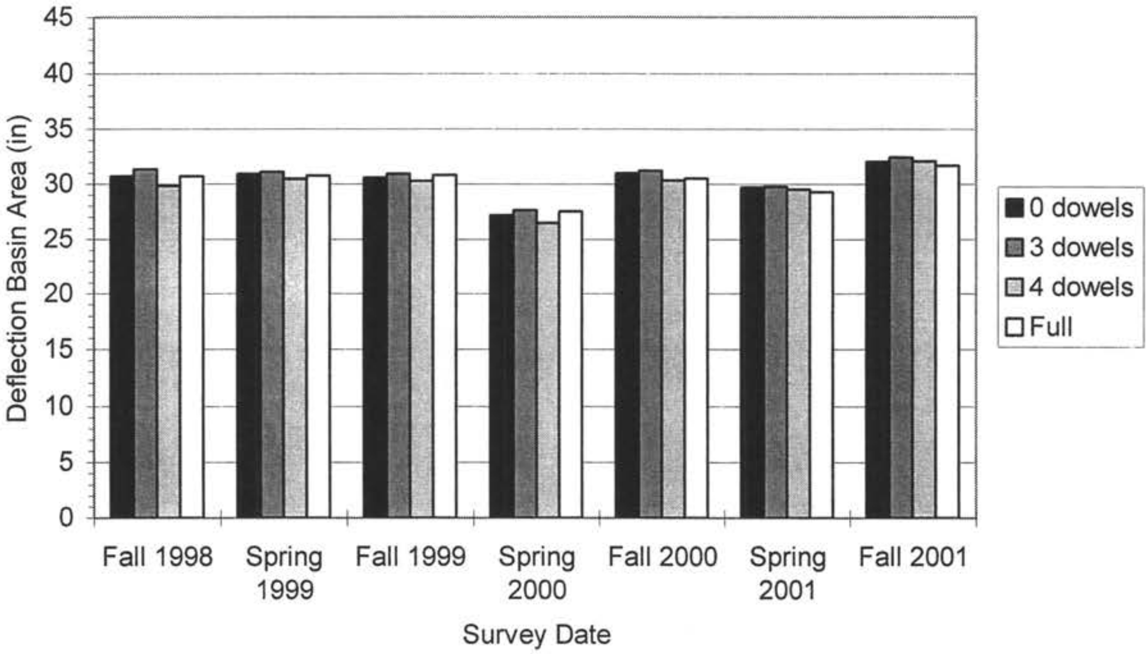


Figure D.3 Average deflection basin area for Urban site westbound lane.

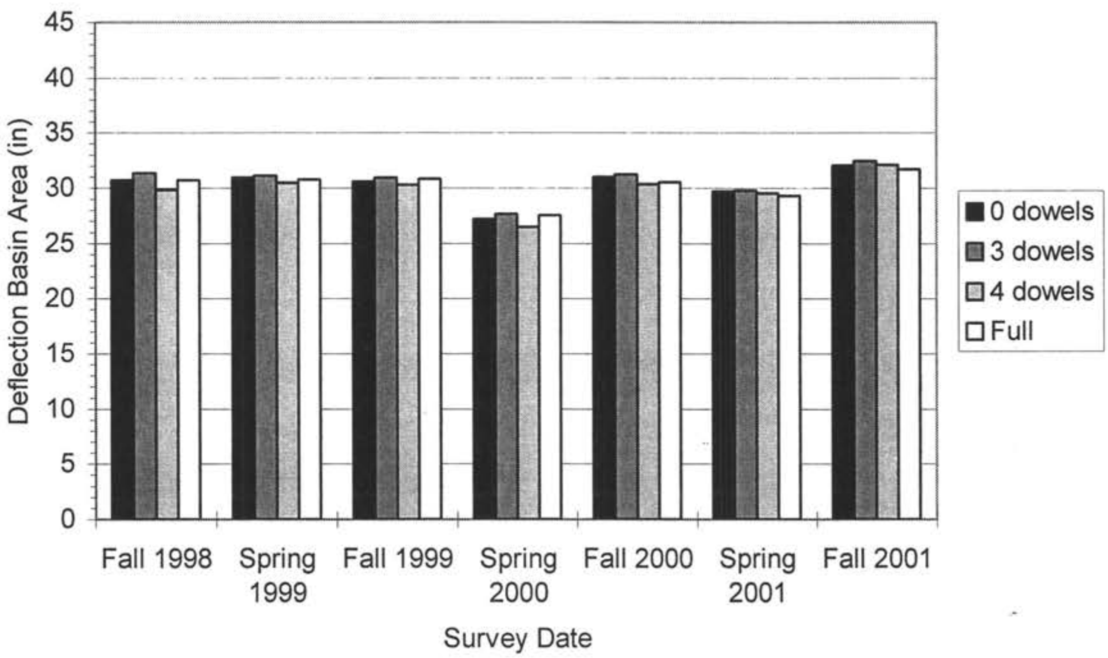


Figure D.4 Average deflection basin area for Urban site eastbound lane.

APPENDIX E: DYNAMIC MODULUS OF SUBGRADE REACTION

E.1 Inside Wheel Path

Table E.1 Average dynamic k-value, Rural site southbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	218.83	215.75	202.15	225.00	221.60	214.80	208.60
3 dowels	209.50	208.50	204.60	222.85	219.10	210.45	205.25
4 dowels	224.75	219.45	159.20	221.55	220.00	216.95	218.70
Full	220.00	213.50	207.10	220.80	223.75	213.45	213.75

Table E.2 Average dynamic k-value, Rural site northbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	218.45	206.00	205.00	221.95	215.35	212.50	212.65
3 dowels	206.50	204.50	198.10	221.55	213.10	208.20	208.15
4 dowels	224.35	213.40	181.60	225.00	222.00	216.65	217.55
Full	218.75	206.75	201.95	223.85	222.10	207.50	214.00

Table E.3 Average dynamic k-value, Urban site westbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	221.00	224.80	223.00	224.80	214.70	222.60	219.00
3 dowels	224.82	225.00	225.00	225.00	222.36	222.91	219.00
4 dowels	223.60	225.00	225.00	225.00	219.10	221.90	217.71
Full	224.20	225.00	225.00	225.00	224.10	224.50	215.43

Table E.4 Average dynamic k-value,Urban site eastbound lane(in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	216.70	222.20	220.00	222.00	205.20	218.20	202.10
3 dowels	223.73	223.00	221.09	225.00	220.82	222.36	212.09
4 dowels	221.50	220.80	223.60	221.40	220.10	221.10	209.20
Full	223.20	223.10	224.80	220.30	222.70	216.40	206.63

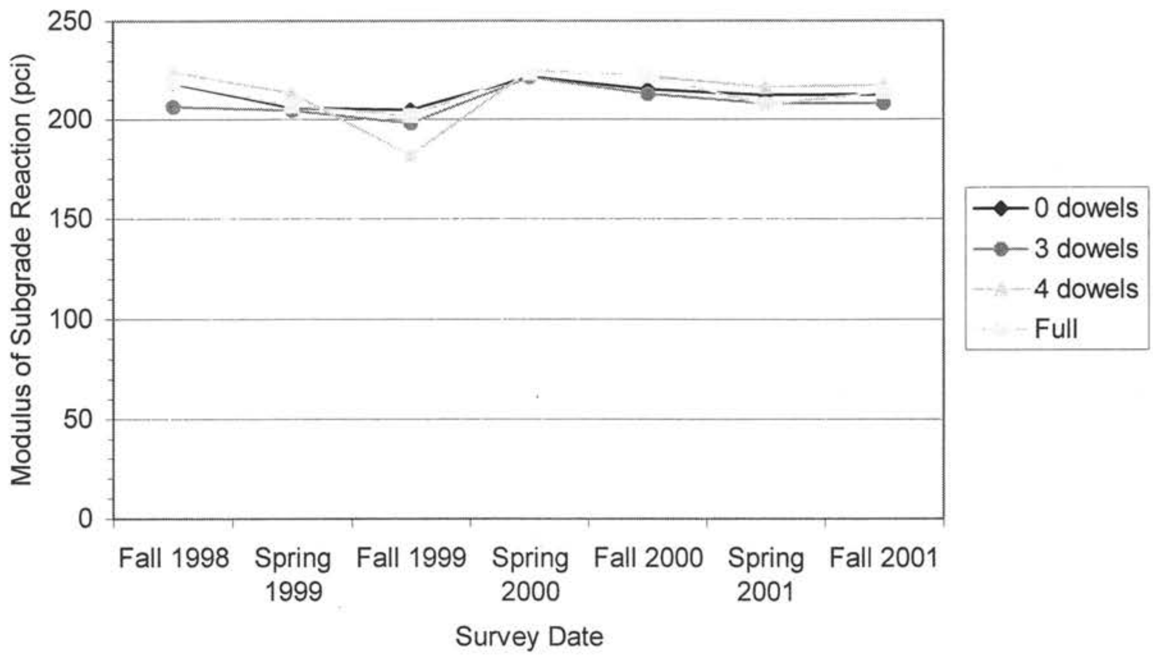


Figure E.1 Average dynamic modulus of subgrade reaction, Rural site southbound lane.

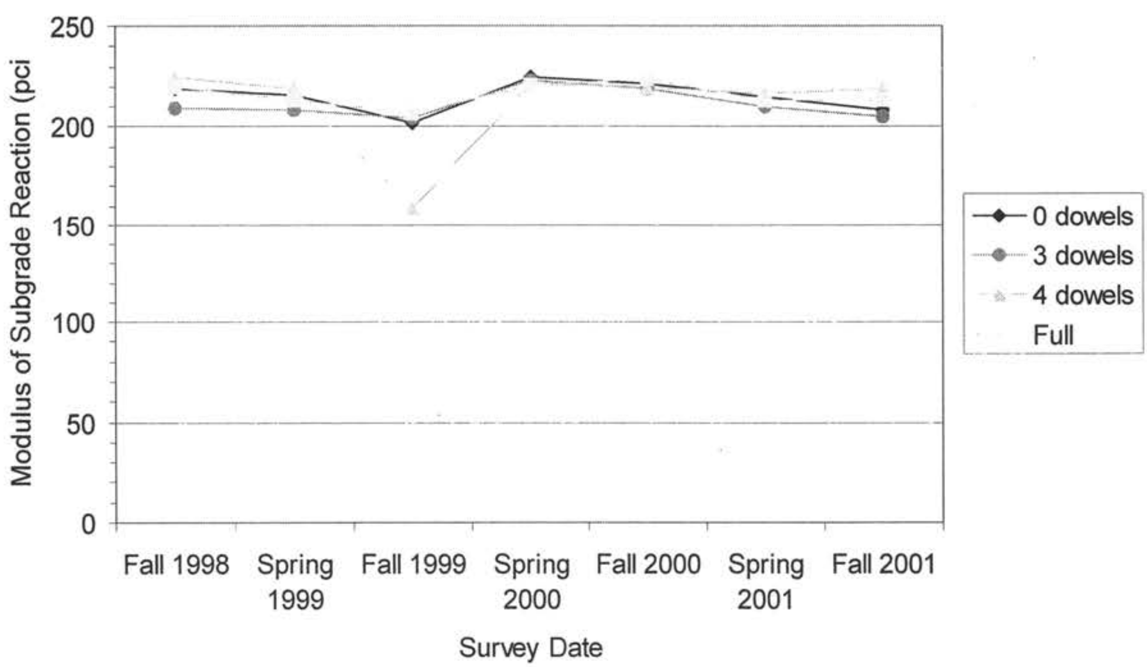


Figure E.2 Average modulus of subgrade reaction, Rural site northbound lane.

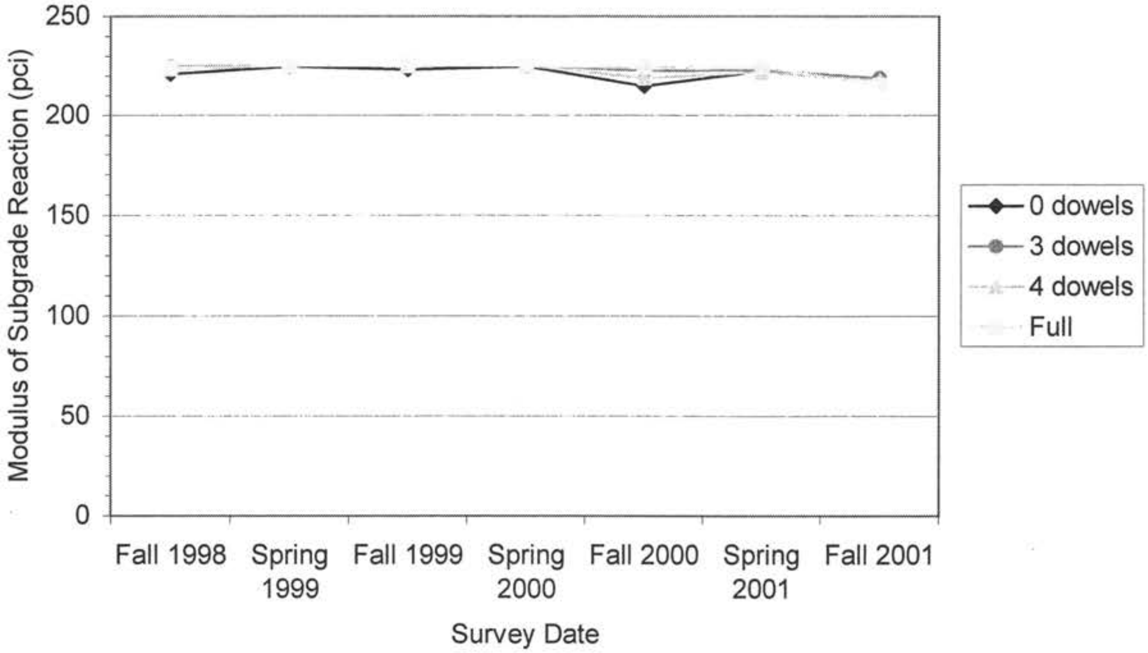


Figure E.3 Average dynamic modulus of subgrade reaction, Urban site westbound lane.

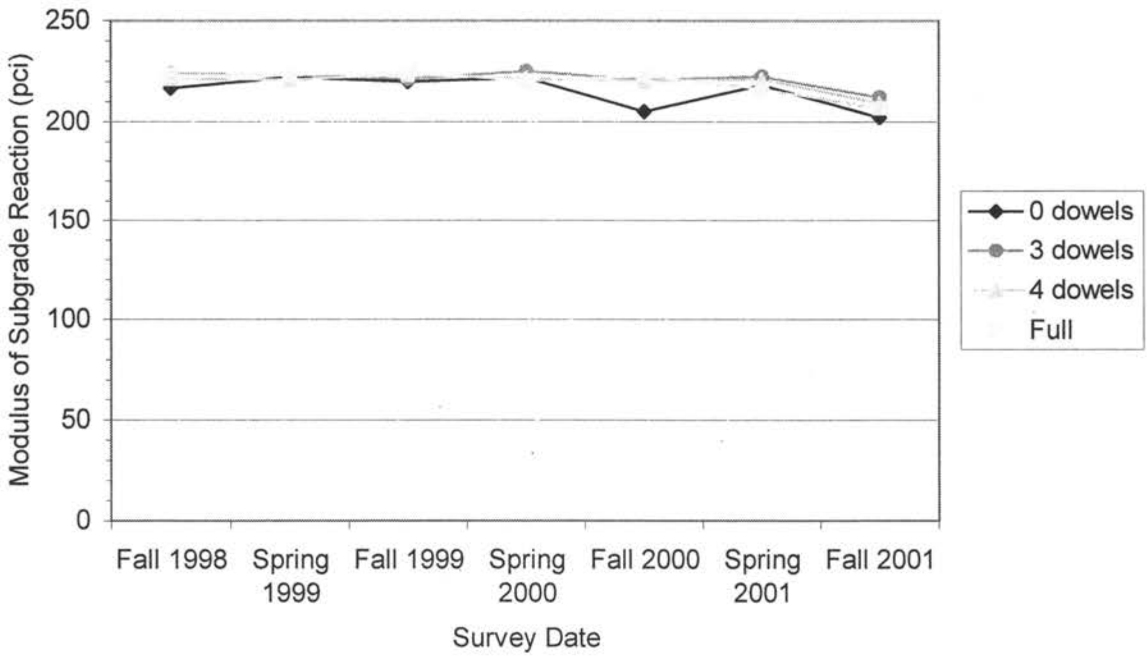


Figure E.4 Average dynamic modulus of subgrade reaction, Urban site eastbound lane.

E.2 Outside Wheel Path

Table E.5 Average dynamic k-value, Rural site southbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	210.93	189.08	197.15	183.15	172.65	145.30	78.25
3 dowels	N.A.	179.65	186.30	194.34	159.30	151.60	58.80
4 dowels	N.A.	187.60	191.75	207.50	193.00	140.85	51.75
Full	N.A.	190.05	193.15	208.05	186.45	176.55	119.45

Table E.6 Average dynamic k-value, Rural site northbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	N.A.	182.50	159.40	75.30	167.50	121.05	55.80
3 dowels	N.A.	163.80	142.15	136.80	120.30	87.60	59.80
4 dowels	N.A.	169.35	141.25	118.15	131.05	57.95	50.00
Full	N.A.	190.33	199.95	218.80	185.30	153.40	111.05

Table E.7 Average dynamic k-value, Urban site westbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	210.00	215.90	214.80	219.40	201.20	200.50	193.00
3 dowels	214.82	222.73	213.45	223.73	199.00	204.64	188.91
4 dowels	213.20	219.90	211.70	222.90	204.10	205.80	191.20
Full	213.50	222.90	220.00	221.40	210.30	213.90	197.90

Table E.8 Average dynamic k-value, Urban site eastbound lane (in pci).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	201.10	203.90	183.30	215.90	184.00	191.20	180.10
3 dowels	214.64	204.73	205.73	215.91	189.00	210.09	177.45
4 dowels	213.20	207.40	198.60	216.50	193.30	200.60	178.80
Full	213.50	205.50	208.00	211.10	193.10	208.60	172.20

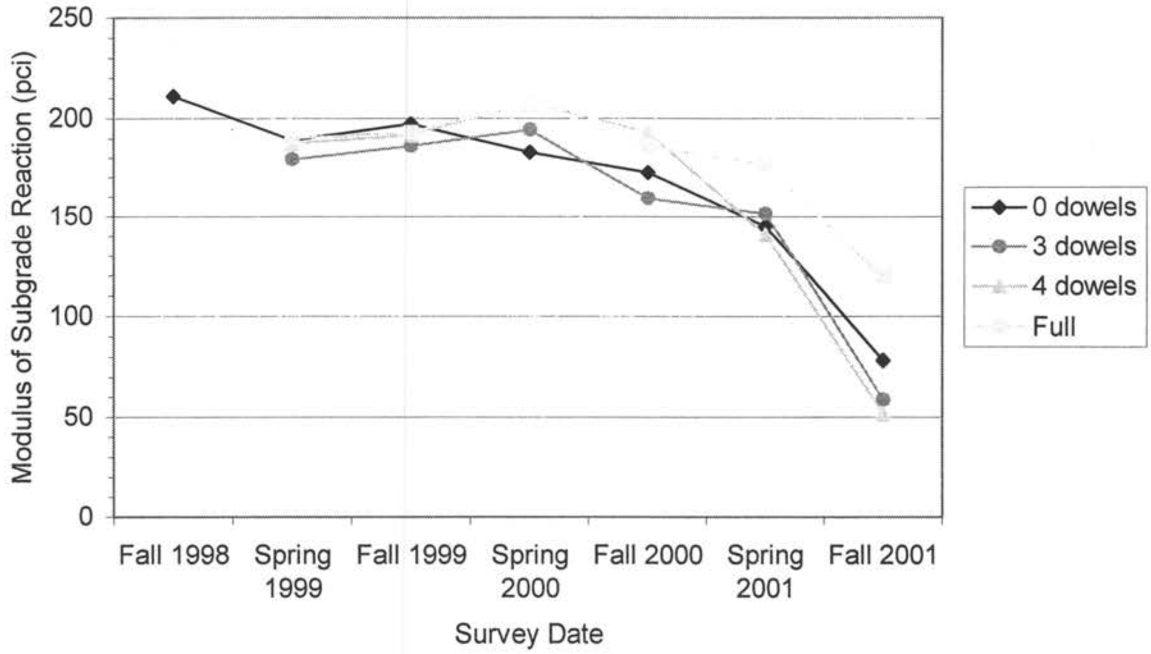


Figure E.5 Average modulus of subgrade reaction for Rural site southbound lane.

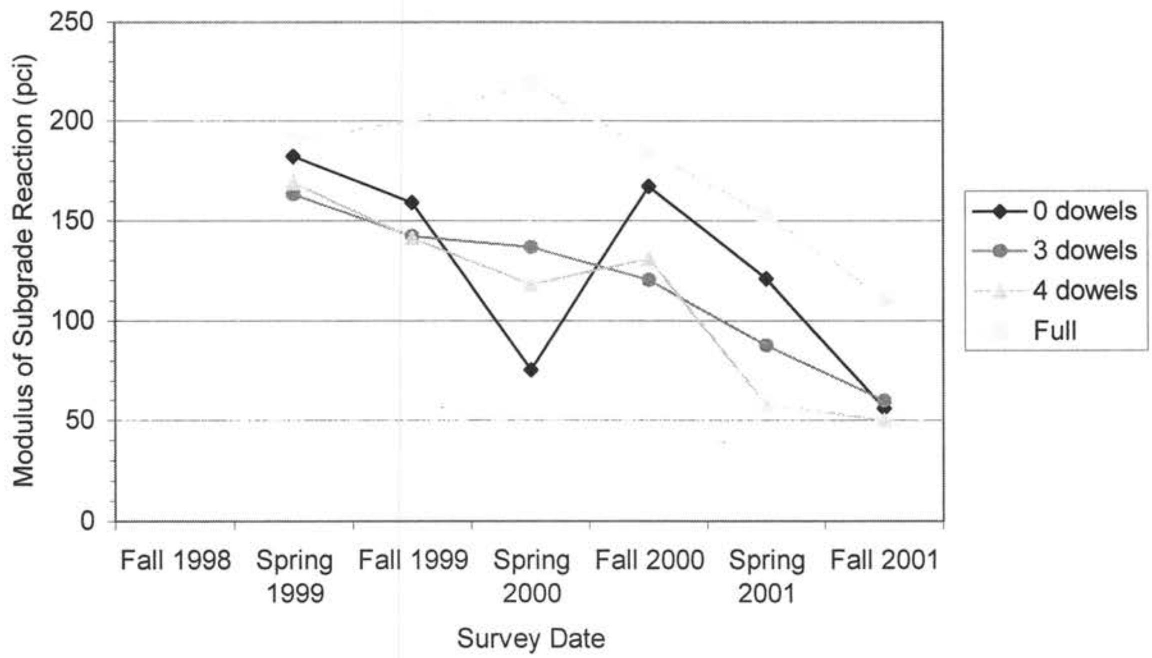


Figure E.6 Average modulus of subgrade reaction for Rural site northbound lane.

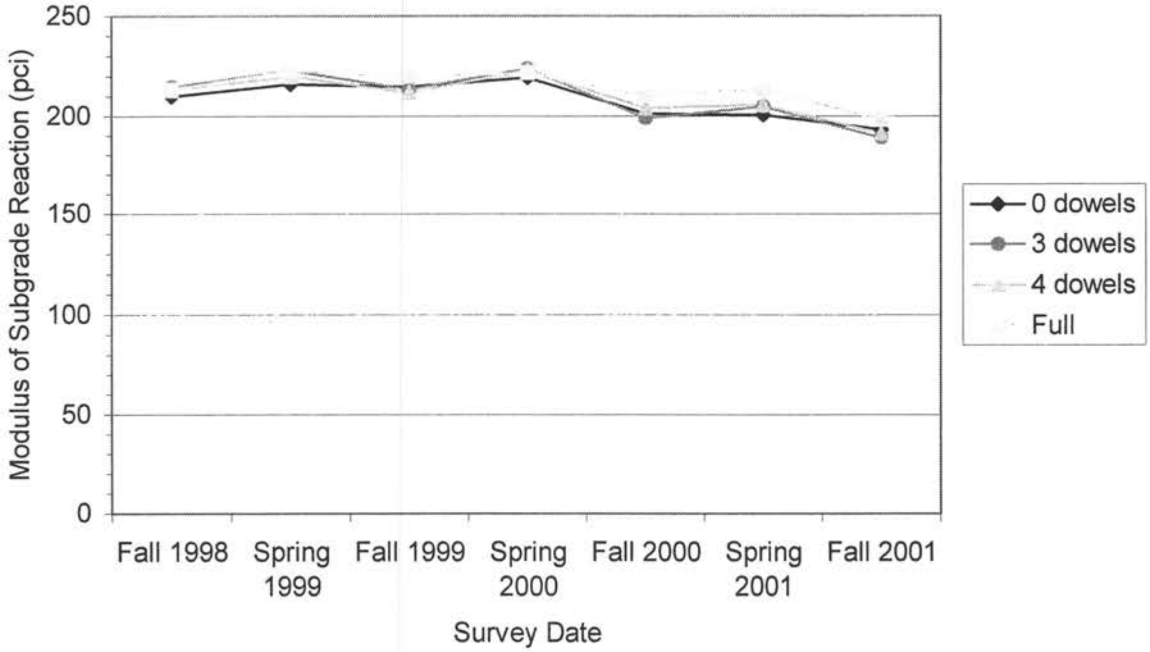


Figure E.7 Average modulus of subgrade reaction for Urban site westbound lane.

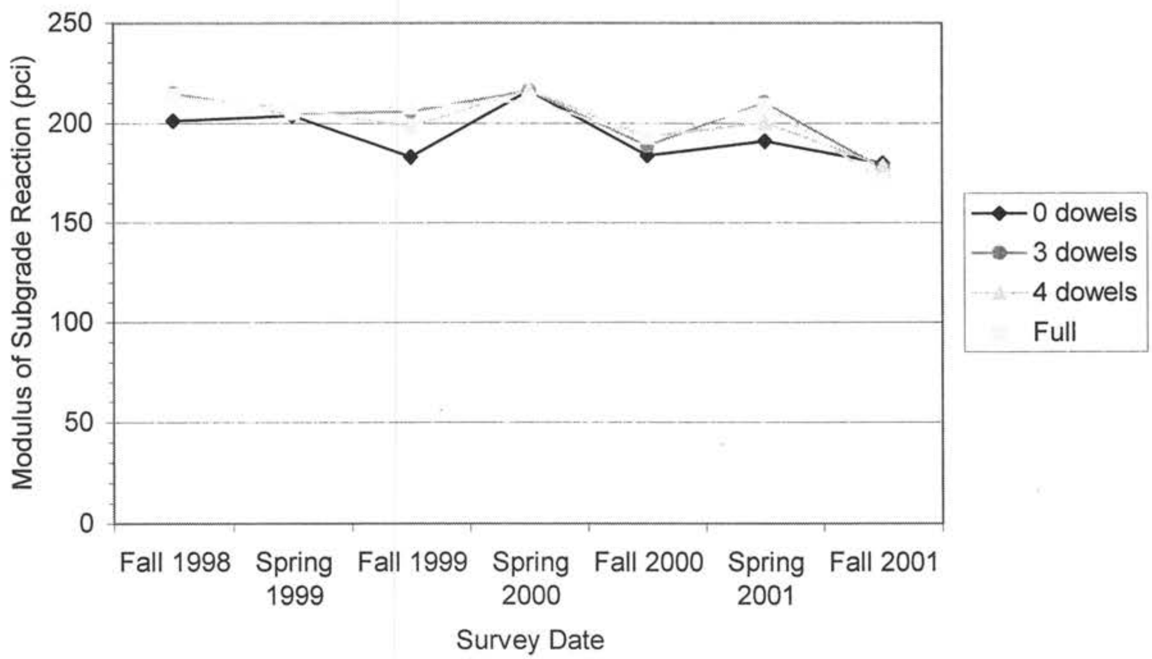


Figure E.8 Average modulus of subgrade reaction for Urban site eastbound lane.

APPENDIX F: MAXIMUM JOINT DEFLECTION

F.1 Inside Wheel Path

Table F.1 Average maximum deflection for Rural site southbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	2.52	1.52	1.34	1.79	2.66	NA	1.45
3 dowels	3.04	1.84	1.62	2.16	2.72	NA	1.54
4 dowels	3.19	1.82	1.95	1.96	2.32	NA	1.76
Full	1.89	1.56	1.40	1.52	2.19	NA	1.52

Table F.2 Average maximum deflection for Rural site northbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	2.73	1.70	1.42	2.12	3.24	NA	4.53
3 dowels	2.84	2.03	1.86	2.75	3.19	NA	2.46
4 dowels	2.47	1.88	1.96	2.75	3.13	NA	3.17
Full	2.62	1.33	1.51	1.62	2.42	NA	1.71

Table F.3 Average maximum deflection for Urban site westbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	0.94	1.18	1.12	1.45	1.26	NA	1.17
3 dowels	0.96	1.04	1.17	1.36	1.28	NA	1.02
4 dowels	0.90	0.96	1.05	1.05	1.16	NA	1.00
Full	0.85	0.84	1.03	0.99	1.01	NA	1.05

Table F.4 Average maximum deflection for Urban site eastbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	0.91	0.88	0.94	1.17	1.14	NA	1.22
3 dowels	0.83	0.88	0.88	1.16	1.39	NA	1.19
4 dowels	0.84	1.04	0.80	1.28	1.21	NA	1.12
Full	0.90	0.77	0.82	1.09	1.14	NA	1.00

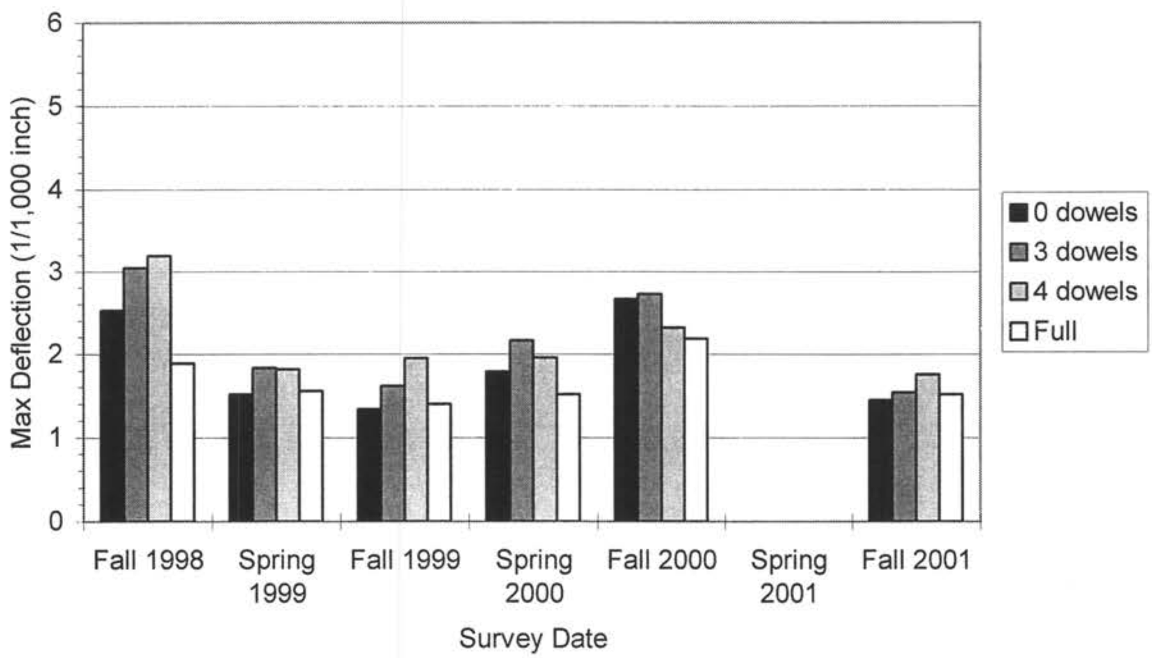


Figure F.1 Average maximum deflection for Rural site southbound lane.

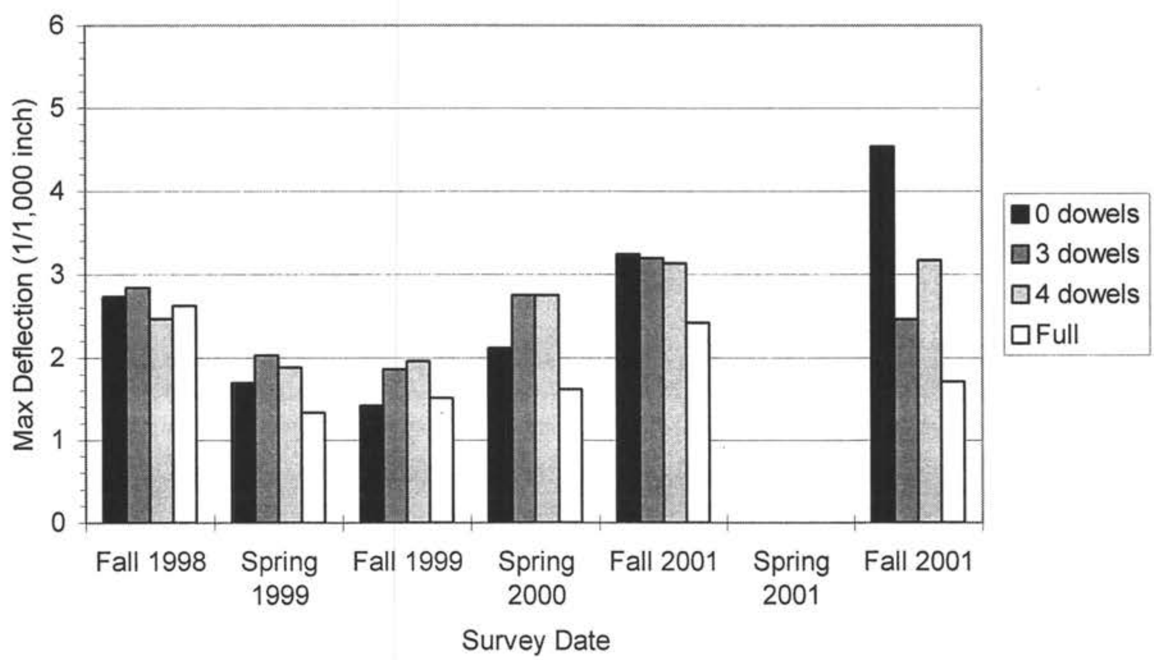


Figure F.2 Average maximum deflection for Rural site northbound lane.

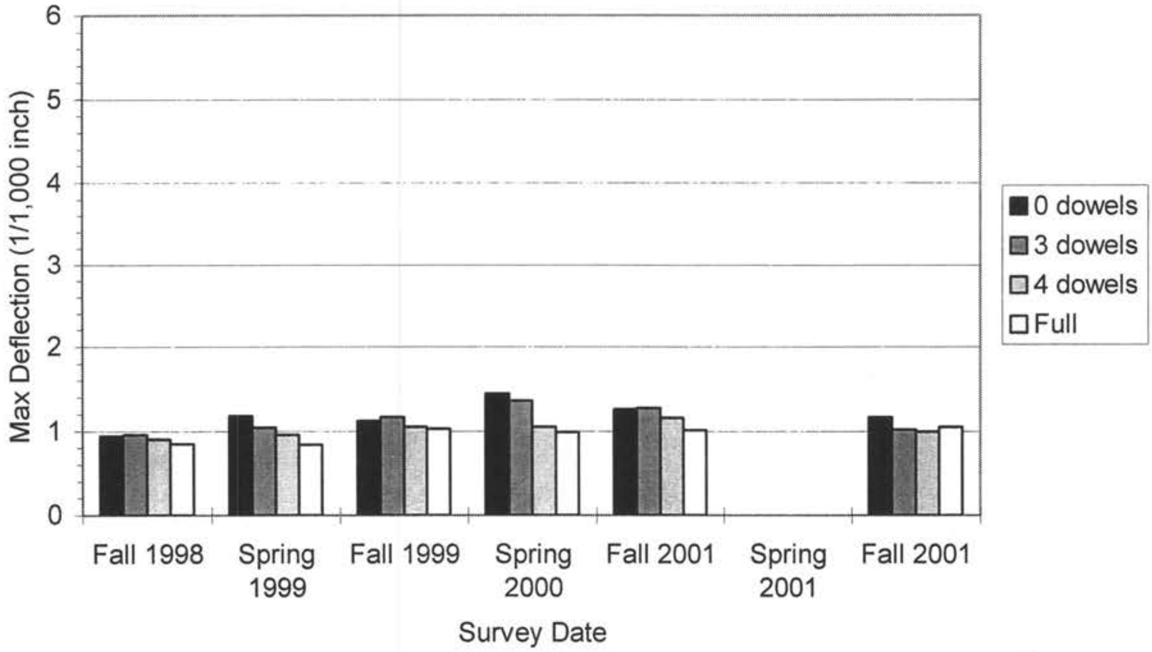


Figure F.3 Average maximum deflection for Urban site westbound lane.

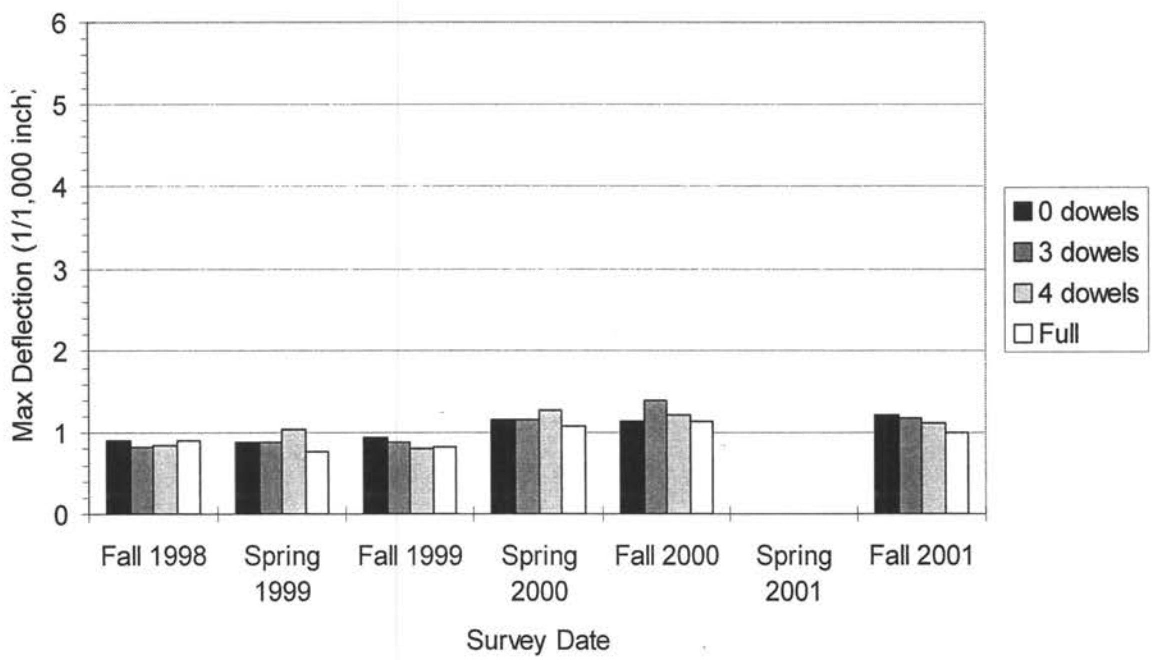


Figure F.4 Average maximum deflection for Urban site eastbound lane.

F.2 Outside Wheel Path

Table F.5 Average maximum deflection for Rural site southbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	2.20	2.16	2.71	5.91	5.37	NA	9.99
3 dowels	NA	2.18	1.76	2.35	3.19	NA	3.85
4 dowels	NA	1.92	1.57	2.62	2.36	NA	4.47
Full	NA	1.93	1.99	2.39	2.45	NA	3.66

Table F.6 Average maximum deflection for Rural site northbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	NA	2.75	5.74	9.99	7.33	NA	9.99
3 dowels	NA	2.22	2.90	3.77	3.47	NA	4.29
4 dowels	NA	2.34	3.37	4.13	3.82	NA	6.16
Full	NA	1.55	1.86	2.73	2.47	NA	3.52

Table F.7 Average maximum deflection for Urban site westbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	1.22	1.60	1.13	1.40	1.43	NA	1.28
3 dowels	0.93	1.13	0.99	1.10	1.32	NA	1.14
4 dowels	0.85	1.03	0.85	1.07	1.27	NA	1.16
Full	1.08	1.02	1.01	1.54	1.25	NA	1.24

Table F.8 Average maximum deflection for Urban site eastbound lane (in mils).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	1.01	1.15	1.17	1.58	1.56	NA	1.57
3 dowels	0.97	1.01	1.01	1.18	1.42	NA	1.50
4 dowels	0.96	1.01	0.86	1.28	1.35	NA	1.50
Full	1.03	1.23	0.83	1.12	1.62	NA	1.63

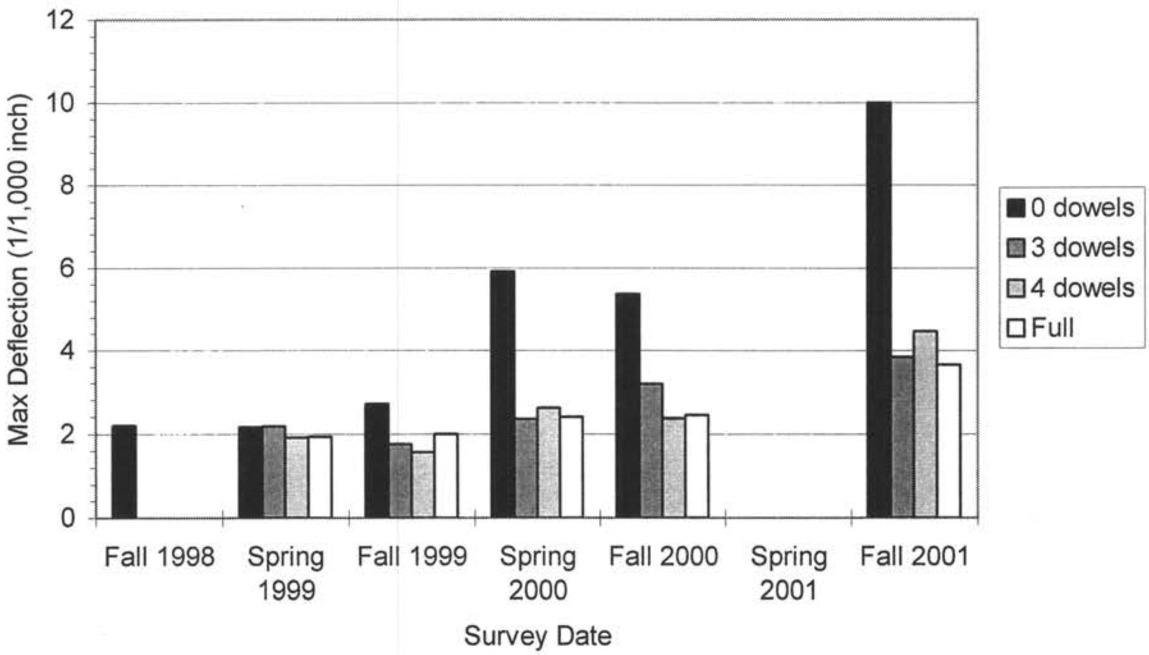


Figure F.5 Average maximum deflection for Rural site southbound lane.

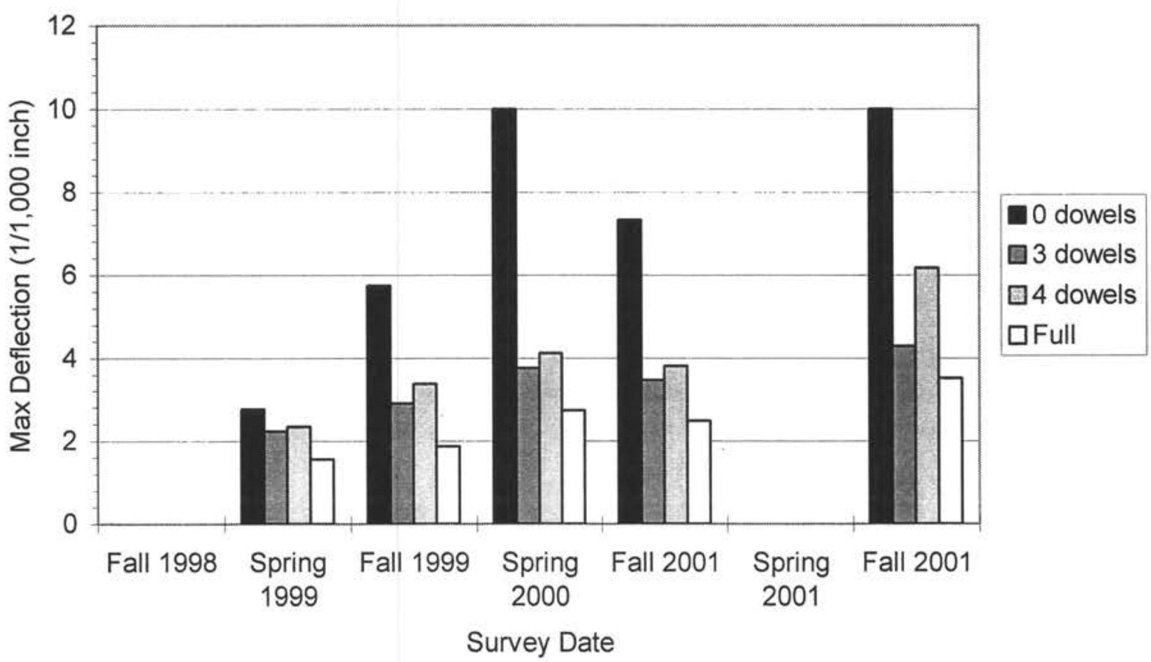


Figure F.6 Average maximum deflection for Rural site northbound lane.

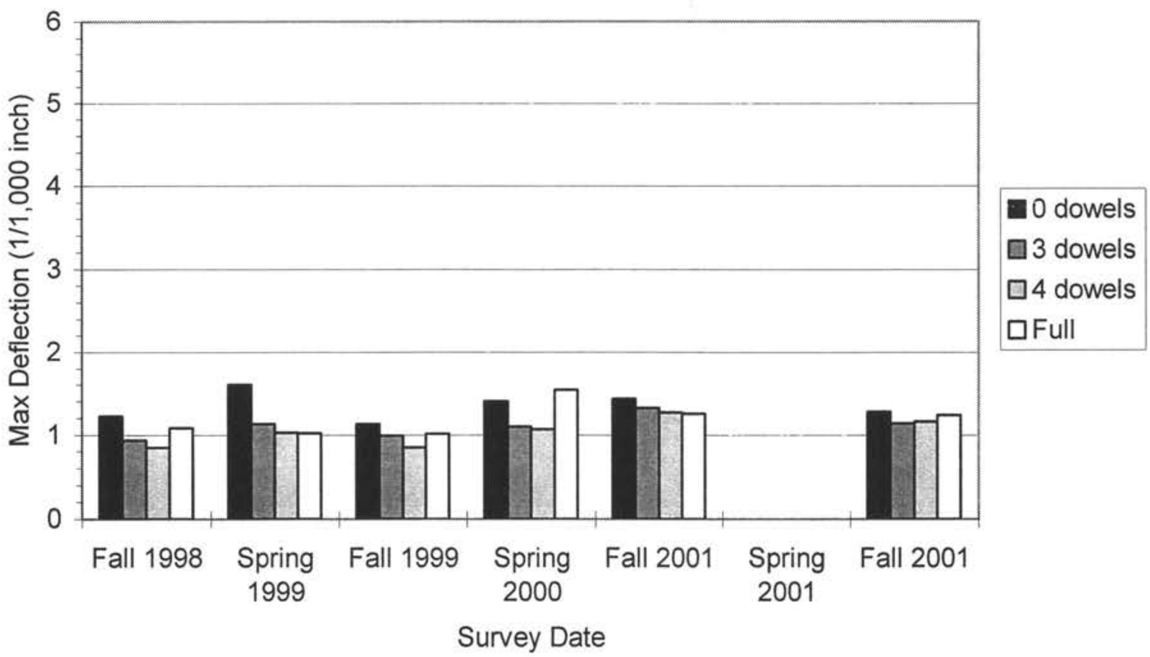


Figure F.7 Average maximum deflection for Urban site westbound lane.

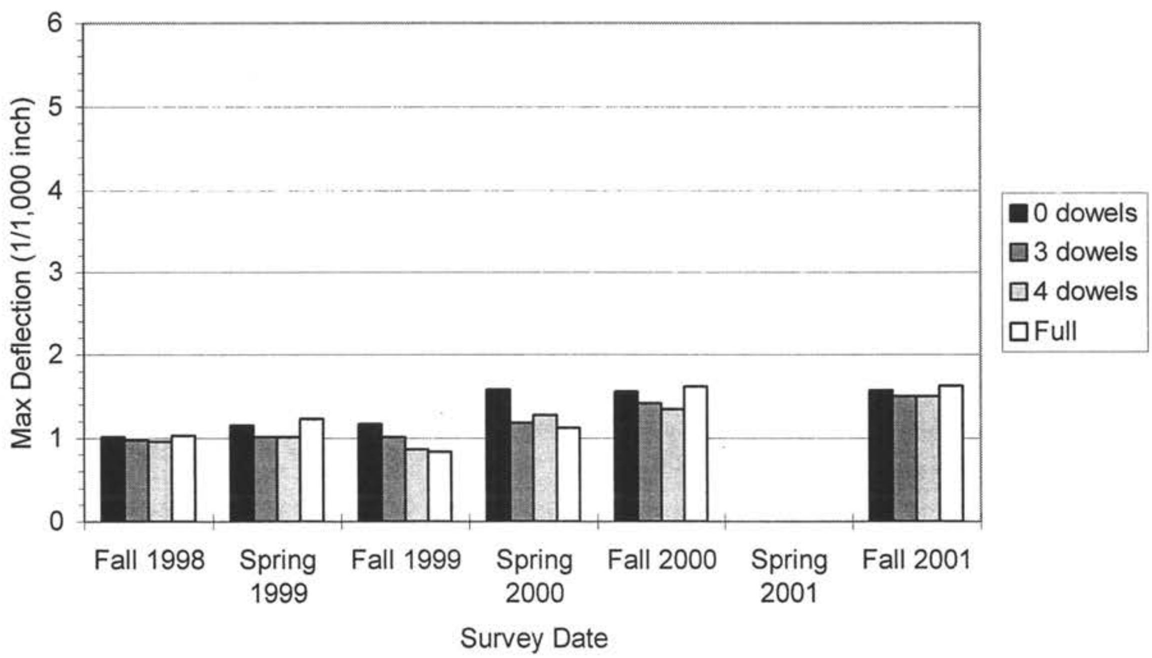


Figure F.8 Average maximum deflection for Urban site eastbound lane.

APPENDIX G: LOAD TRANSFER

G.1 Inside Wheel Path

Table G.1 Average load transfer for Rural site southbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	79.99	84.10	87.82	79.84	34.19	NA	87.04
3 dowels	78.92	82.68	85.95	78.98	38.51	NA	87.78
4 dowels	76.53	81.01	84.70	62.10	32.89	NA	85.09
Full	79.26	83.51	85.16	88.78	72.41	NA	90.17

Table G.2 Average load transfer for Rural site northbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	80.67	83.23	87.96	55.76	26.39	NA	53.96
3 dowels	81.19	82.41	87.15	64.51	30.83	NA	93.07
4 dowels	78.13	78.75	82.61	54.90	27.70	NA	93.73
Full	81.86	85.91	82.09	85.37	64.46	NA	92.43

Table G.3 Average load transfer for Urban site westbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	87.07	85.59	81.83	67.60	63.71	NA	87.80
3 dowels	87.27	84.17	81.32	76.05	61.51	NA	88.90
4 dowels	87.36	85.20	82.29	84.18	63.53	NA	88.21
Full	89.99	86.91	82.27	83.59	83.36	NA	89.77

Table G.4 Average load transfer for Urban site eastbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	90.18	89.38	89.82	88.38	83.38	NA	87.78
3 dowels	88.57	88.15	87.99	77.74	56.01	NA	78.08
4 dowels	89.55	86.73	88.08	74.39	62.67	NA	81.47
Full	89.71	89.78	88.77	86.42	77.45	NA	86.99

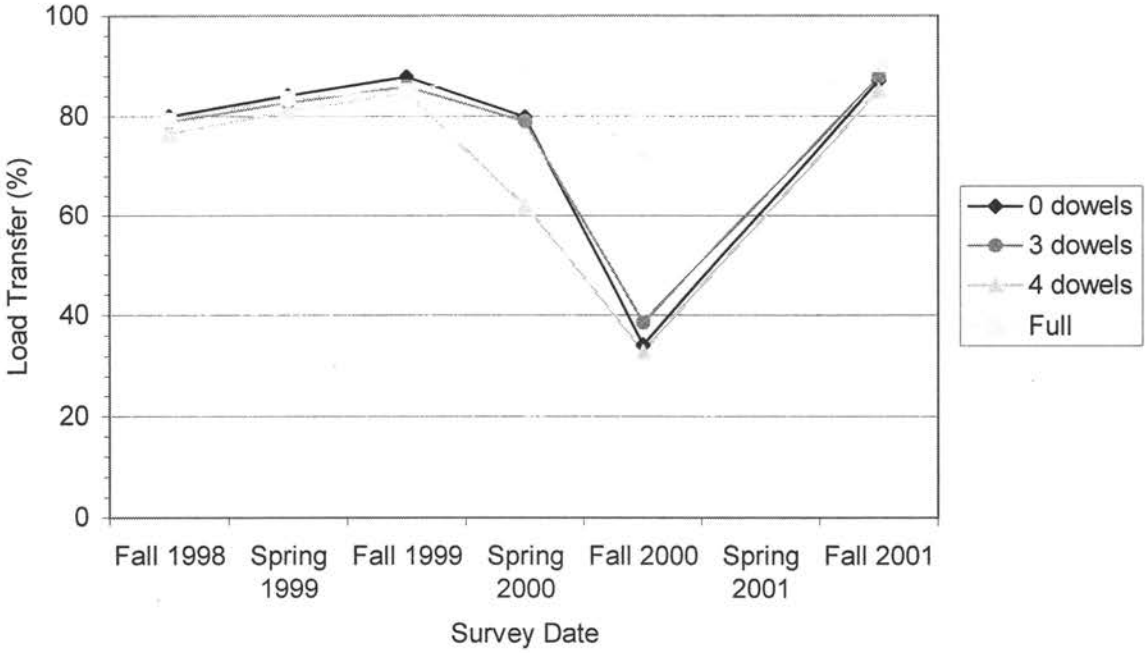


Figure G.1 Average load transfer for Rural site southbound lane.

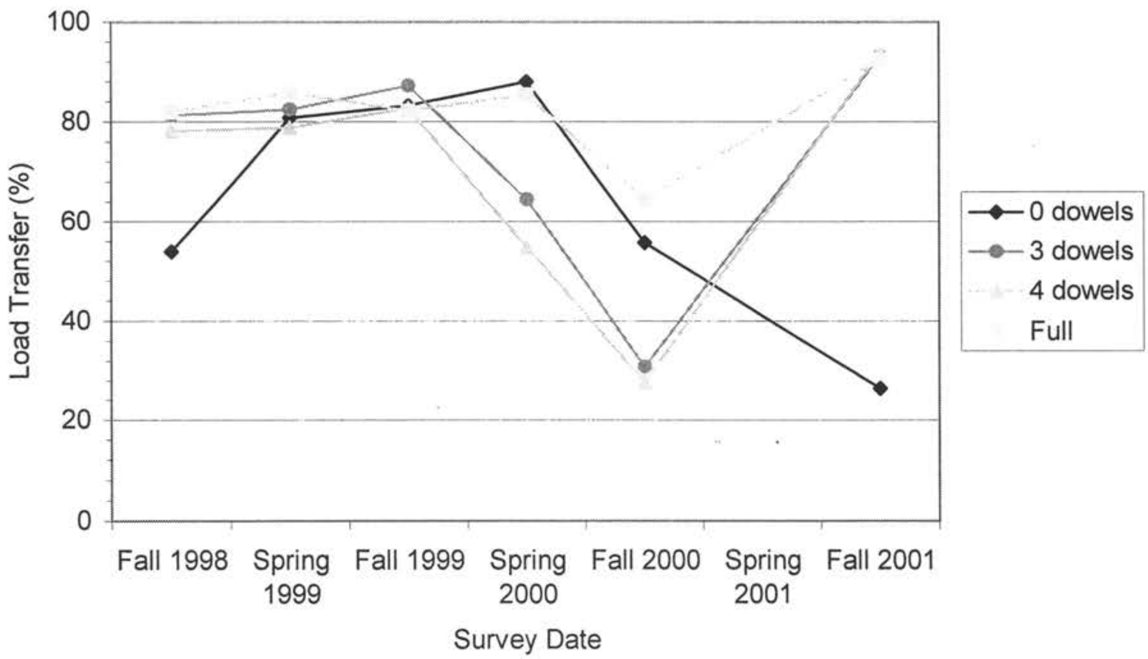


Figure G.2 Average load transfer for Rural site northbound lane.

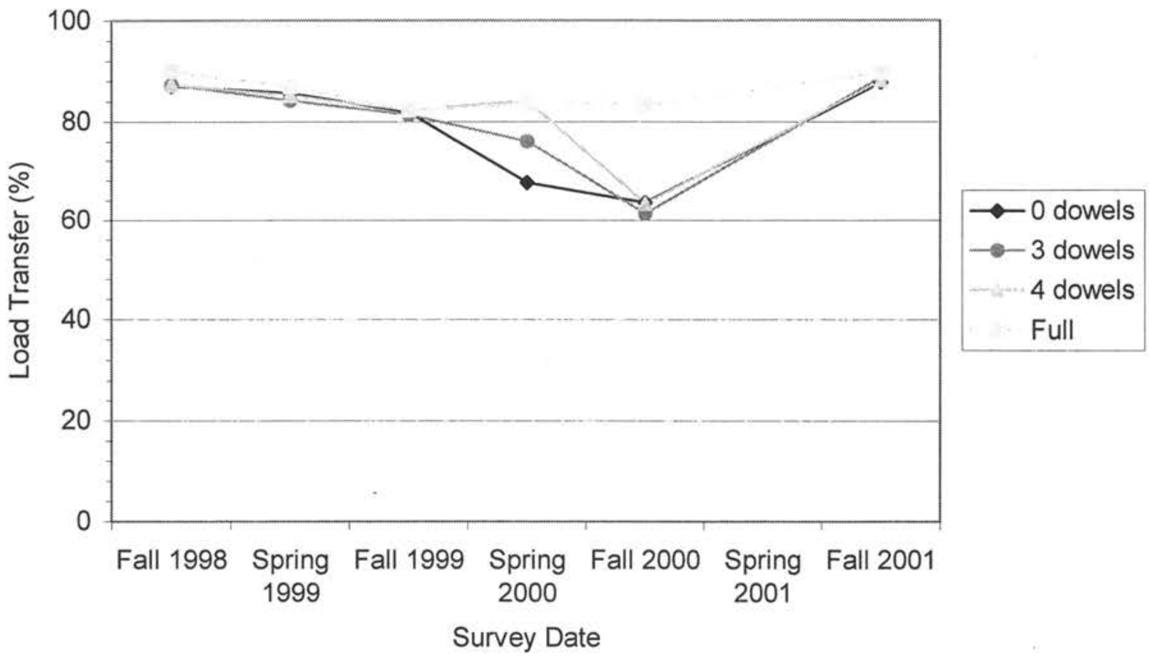


Figure G.3 Average load transfer for Urban site westbound lane.

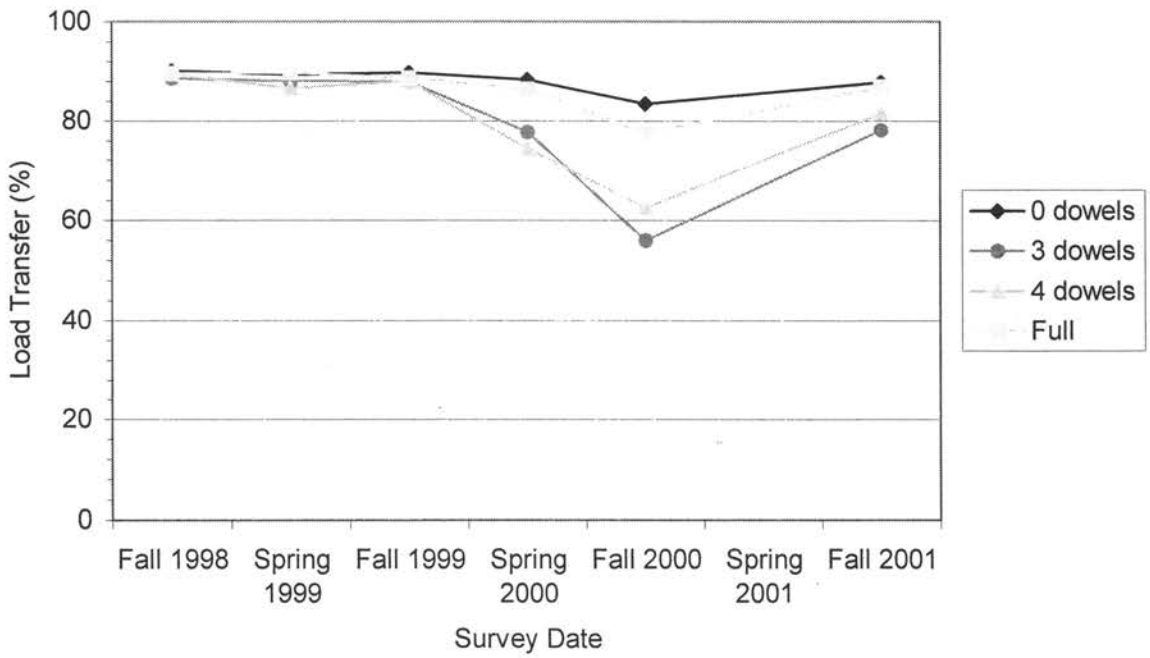


Figure G.4 Average load transfer for Urban site eastbound lane.

G.2 Outside Wheel Path

Table G.5 Average load transfer for Rural site southbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	84.97	90.72	86.70	40.76	29.00	NA	47.26
3 dowels	NA	87.67	86.37	86.63	76.20	NA	74.38
4 dowels	NA	88.14	85.16	80.08	73.39	NA	74.02
Full	NA	87.93	83.68	88.07	84.66	NA	83.64

Table G.6 Average load transfer for Rural site northbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	NA	93.14	81.38	20.86	22.34	NA	44.20
3 dowels	NA	84.95	82.59	75.58	70.57	NA	90.84
4 dowels	NA	87.20	83.93	74.04	70.51	NA	91.43
Full	NA	88.12	81.62	80.63	76.21	NA	90.23

Table G.7 Average load transfer for Urban site westbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2001	Spring 2001	Fall 2001
0 dowels	87.49	88.42	88.96	79.40	67.34	NA	88.61
3 dowels	88.02	89.11	91.35	92.46	83.70	NA	91.80
4 dowels	88.84	91.57	91.59	93.41	88.36	NA	91.00
Full	89.50	91.11	90.13	93.45	87.16	NA	90.38

Table G.8 Average load transfer for Urban site eastbound lane (in %).

	Fall 1998	Spring 1999	Fall 1999	Spring 2000	Fall 2000	Spring 2001	Fall 2001
0 dowels	90.84	89.99	90.47	88.99	86.16	NA	87.80
3 dowels	89.33	89.64	90.81	87.32	79.75	NA	88.90
4 dowels	90.32	91.04	92.39	88.63	85.15	NA	88.21
Full	90.41	87.83	91.30	91.97	88.87	NA	89.77

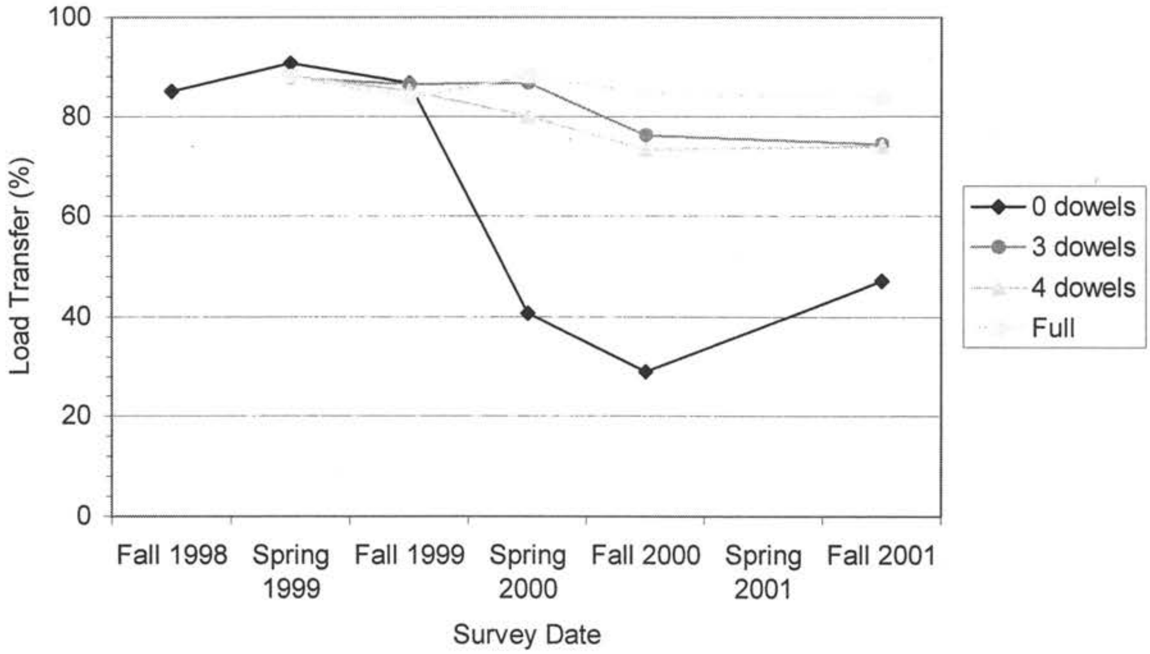


Figure G.5 Average load transfer for Rural site southbound lane.

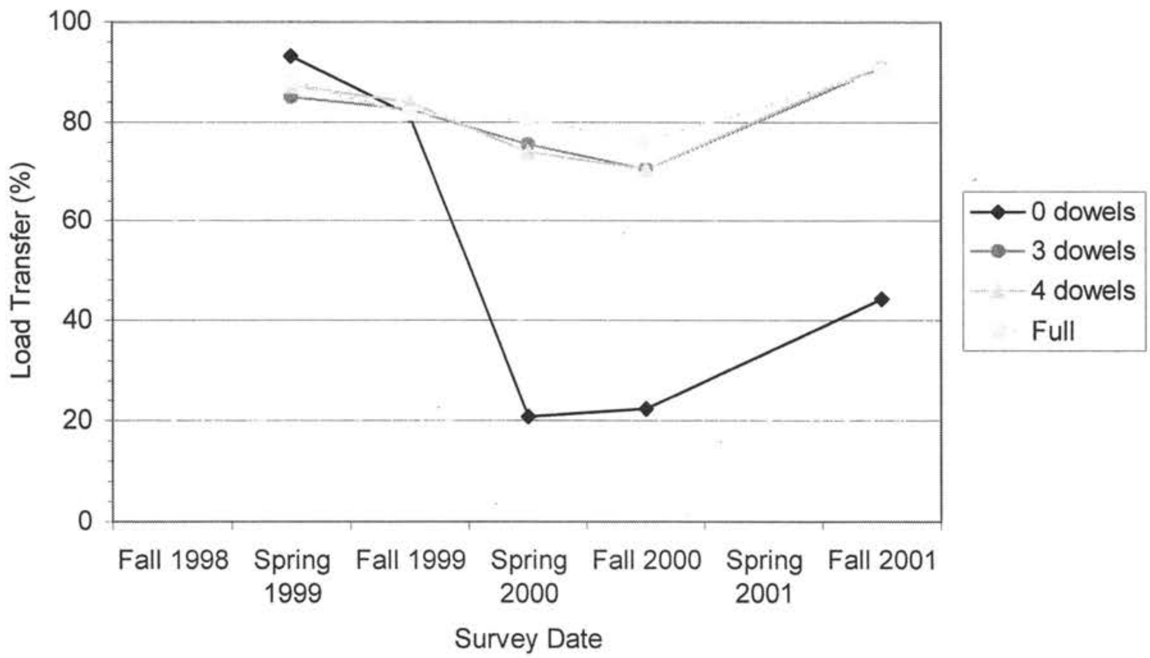


Figure G.6 Average load transfer for Rural site northbound lane.

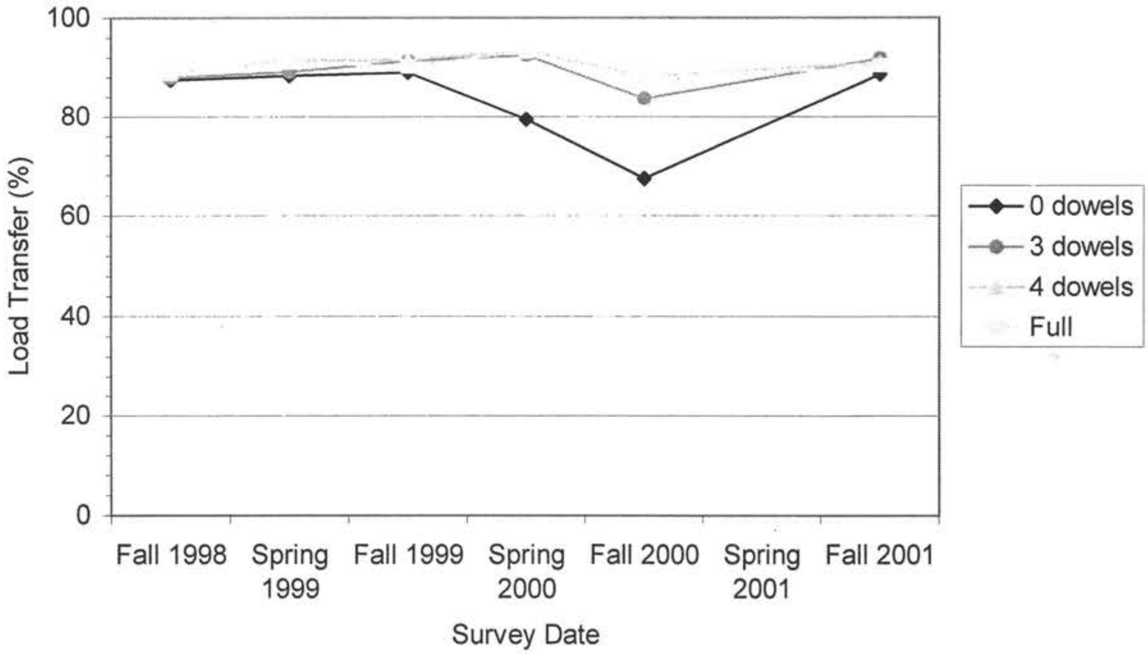


Figure G.7 Average load transfer for Urban site westbound lane.

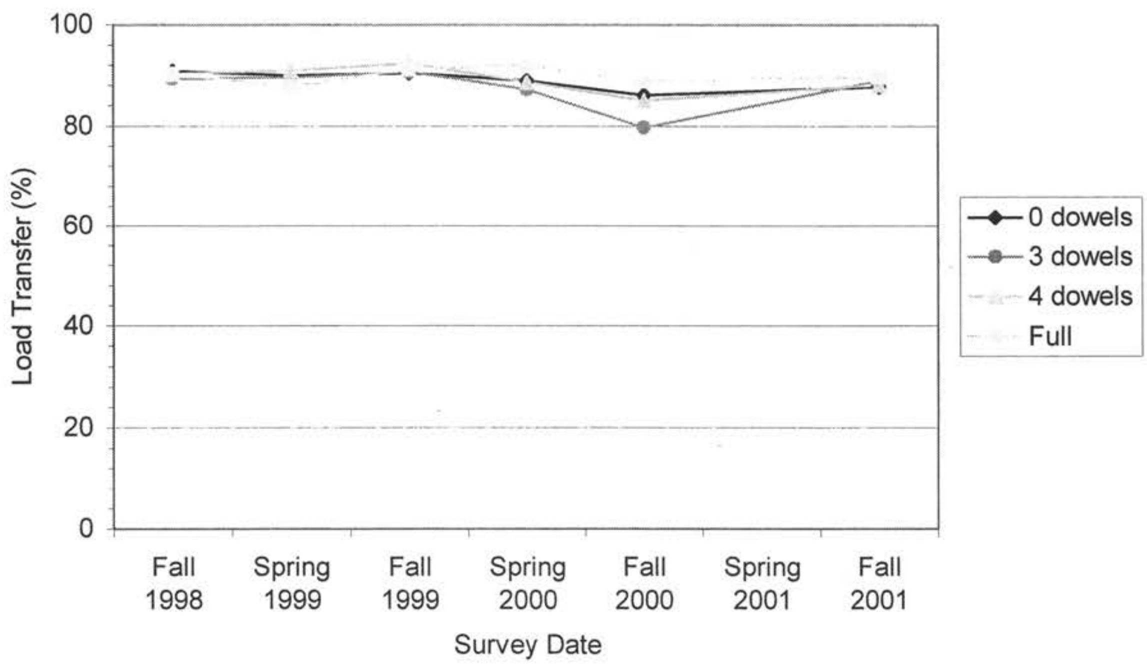


Figure G.8 Average load transfer for Urban site eastbound lane.

G.3 Additional Figures

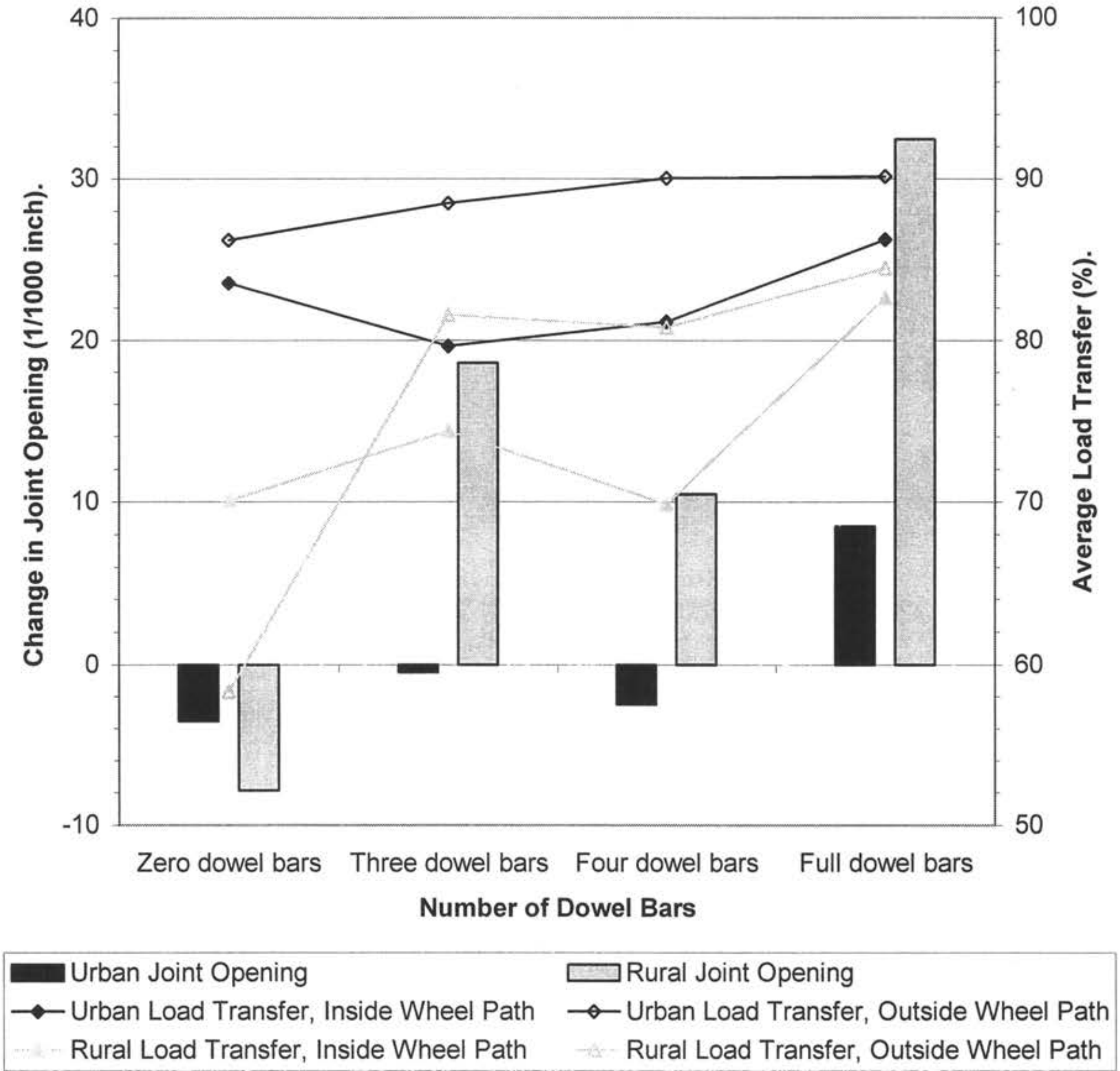


Figure G.9 Average joint opening and average load transfer vs. number of dowel bars.

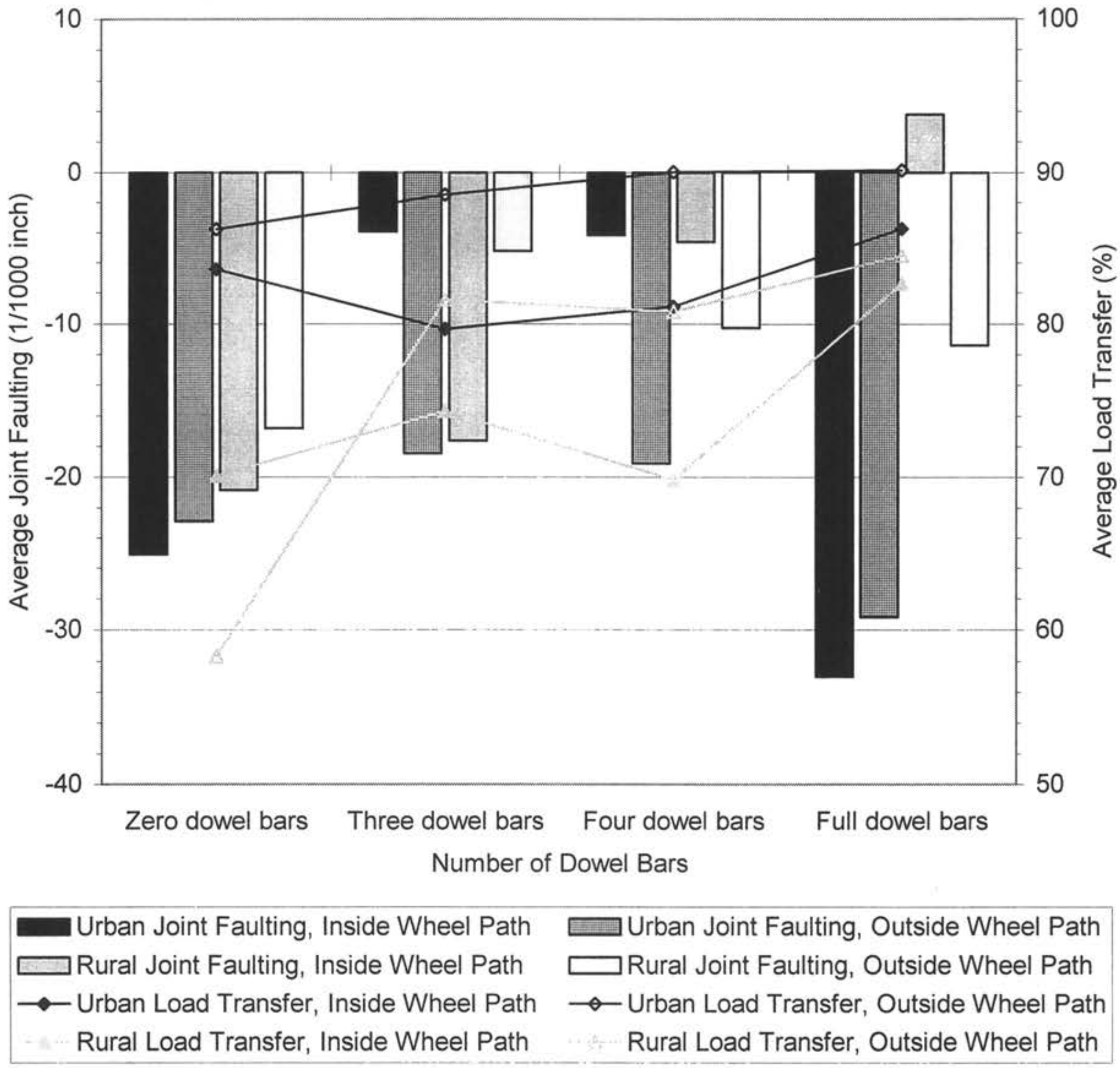


Figure G.10 Average joint faulting and average load transfer vs. number of dowel bars.

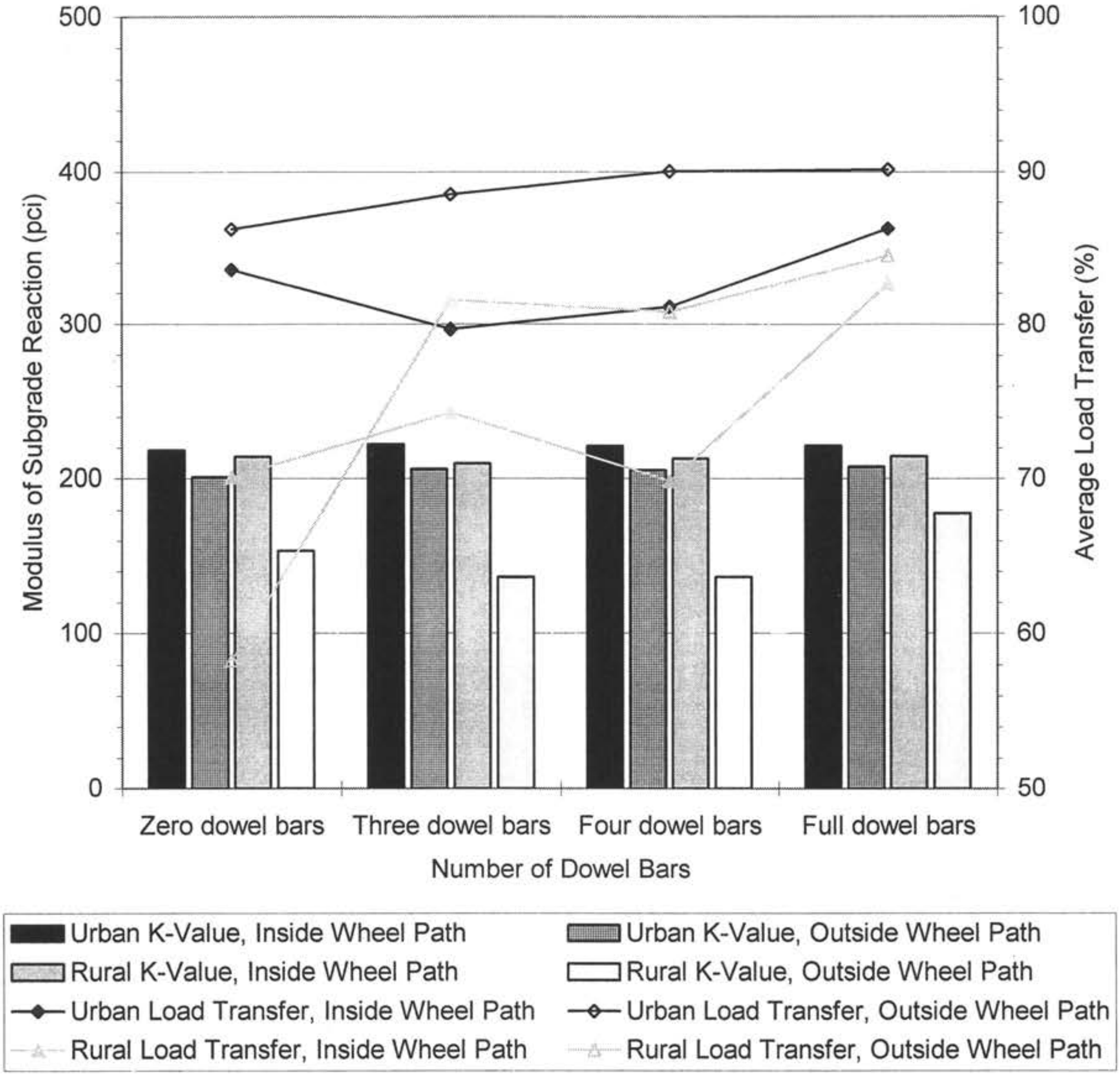


Figure G.11 Average k-value and average load transfer vs. number of dowel bars.

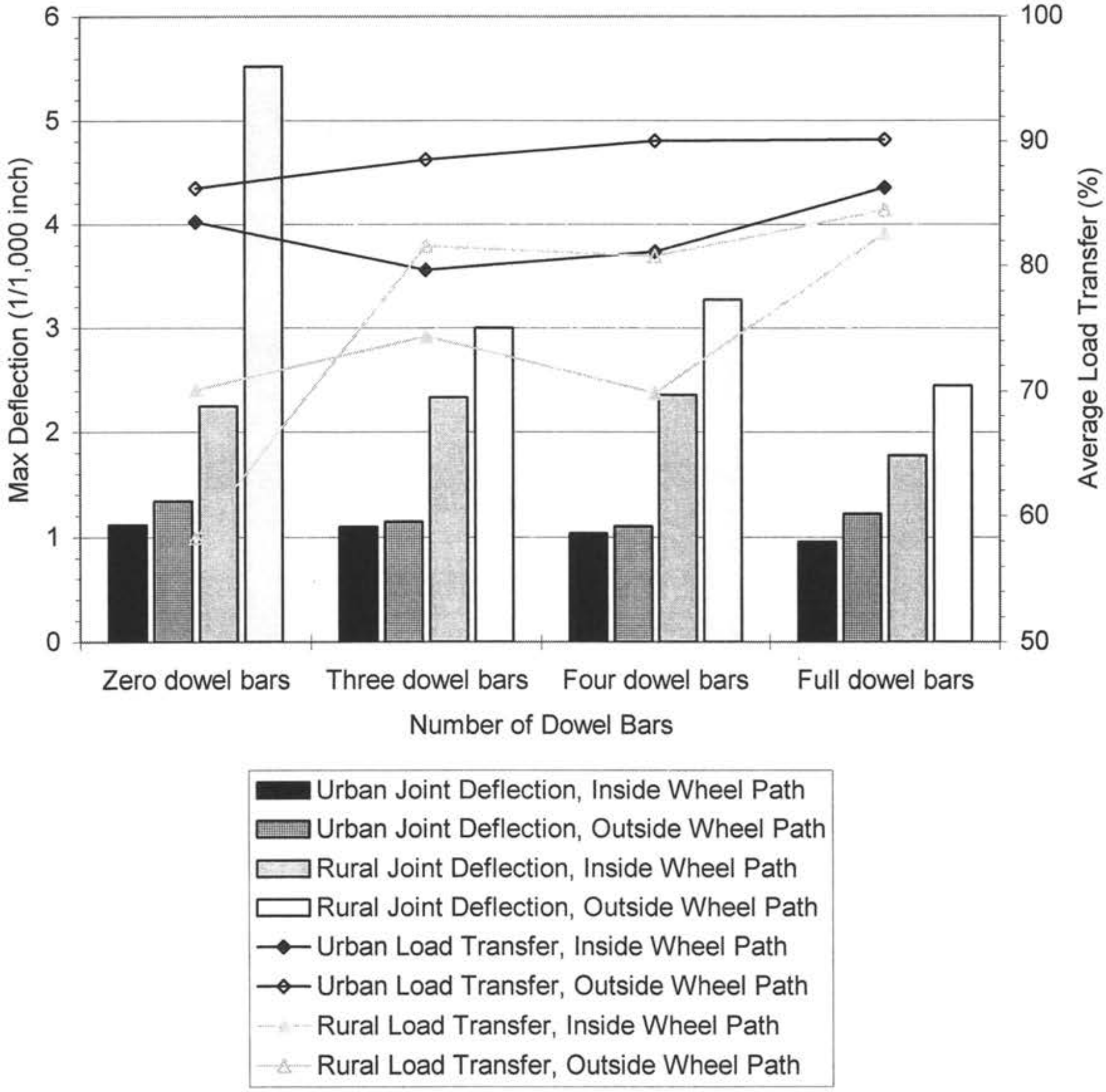


Figure G.12 Average maximum deflection and average load transfer vs. number of dowel bars.

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